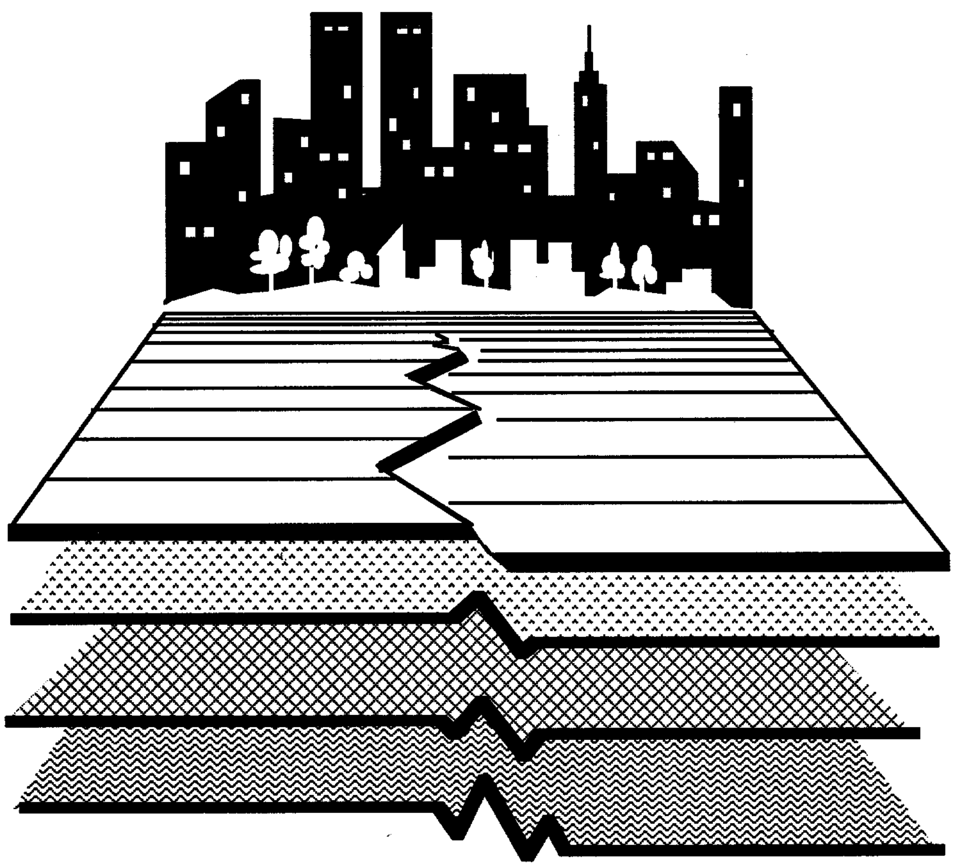
NEHRP GUIDELINES FOR THE SEISMIC REHABILITATION OF BUILDINGS



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

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: (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 general 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.

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BUILDING SEISMIC SAFETY COUNCIL

A council of the National Institute of Building Sciences

BSSC

Seismic

Rehabilitation

Project

NEHRP GUIDELINES FOR THE SEISMIC REHABILITATION OF BUILDINGS (FEMA Publication 273)

Prepared for the
BUILDING SEISMIC SAFETY COUNCIL
Washington, D.C.

By the
APPLIED TECHNOLOGY COUNCIL (ATC-33 Project)
Redwood City, California

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In Memoriam

The Building Seismic Safety Council, the Applied Technology Council, the American Society of Civil Engineers, and the Federal Emergency Management Agency wish to acknowledge the significant contribution to the *Guidelines* and to the overall field of earthquake engineering of the participants in the project who did not live to see this effort completed:

Richard Atkinson

Peter Gergely

Roger Scholl

The built environment has benefited greatly from their work.

Foreword

The volume you are now holding in your hands, the *NEHRP Guidelines for the Seismic Rehabilitation of Buildings*, and its companion *Commentary* volume, are the culminating manifestation of over 13 years of effort. They contain systematic guidance enabling design professionals to formulate effective and reliable rehabilitation approaches that will limit the expected earthquake damage to a specified range for a specified level of ground shaking. This kind of guidance applicable to all types of existing buildings and in all parts of the country has never existed before.

Since 1984, when the Federal Emergency Management Agency (FEMA) first began a program to address the risk posed by seismically unsafe existing buildings, the creation of these *Guidelines* has been the principal target of FEMA's efforts. Prior preparatory steps, however, were much needed, as was noted in the 1985 *Action Plan* developed at FEMA's request by the ABE Joint Venture. These included the development of a standard methodology for identifying at-risk buildings quickly or in depth, a compendium of effective rehabilitation techniques, and an identification of societal implications of rehabilitation.

By 1990, this technical platform had been essentially completed, and work could begin on these *Guidelines*. The $8 million, seven-year project required the varied talents of over 100 engineers, researchers and writers, smoothly orchestrated by the Building Seismic Safety Council (BSSC), overall manager of the project; the Applied Technology Council (ATC); and the American Society of Civil Engineers (ASCE). Hundreds more donated their knowledge and time to the project by reviewing draft documents at various stages of development and providing comments, criticisms, and suggestions for improvements. Additional refinements and improvements resulted from the consensus review of the *Guidelines* document and its companion *Commentary* through the balloting process of the BSSC during the last year of the effort.

No one who worked on this project in any capacity, whether volunteer, paid consultant or staff, received monetary compensation commensurate with his or her efforts. The dedication of all was truly outstanding. It seemed that everyone involved recognized the magnitude of the step forward that was being taken in the progress toward greater seismic safety of our communities, and gave his or her utmost. FEMA and the FEMA Project Officer personally warmly and sincerely thank everyone who participated in this endeavor. Simple thanks from FEMA in a Foreword, however, can never reward these individuals adequately. The fervent hope is that, perhaps, having the *Guidelines* used extensively now and improved by future generations will be the reward that they so justly and richly deserve.

The Federal Emergency Management Agency

Preface

In August 1991, the National Institute of Building Sciences (NIBS) entered into a cooperative agreement with the Federal Emergency Management Agency (FEMA) for a comprehensive seven-year program leading to the development of a set of nationally applicable guidelines for the seismic rehabilitation of existing buildings. Under this agreement, the Building Seismic Safety Council (BSSC) served as program manager with the American Society of Civil Engineers (ASCE) and the Applied Technology Council (ATC) working as subcontractors. Initially, FEMA provided funding for a program definition activity designed to generate the detailed work plan for the overall program. The work plan was completed in April 1992 and in September FEMA contracted with NIBS for the remainder of the effort.

The major objectives of the project were to develop a set of technically sound, nationally applicable guidelines (with commentary) for the seismic rehabilitation of buildings; develop building community consensus regarding the guidelines; and develop the basis of a plan for stimulating widespread acceptance and application of the guidelines. The guidelines documents produced as a result of this project are expected to serve as a primary resource on the seismic rehabilitation of buildings for the use of design professionals, educators, model code and standards organizations, and state and local building regulatory personnel.

As noted above, the project work involved the ASCE and ATC as subcontractors as well as groups of volunteer experts and paid consultants. It was structured to ensure that the technical guidelines writing effort benefited from a broad section of considerations: the results of completed and ongoing technical efforts and research activities; societal issues; public policy concerns; the recommendations presented in an earlier FEMA-funded report on issues identification and resolution; cost data on application of rehabilitation procedures; reactions of potential users; and consensus review by a broad spectrum of building community interests. A special effort also was made to use the results of the latest relevant research.

While overall management has been the responsibility of the BSSC, responsibility for conduct of the specific project tasks is shared by the BSSC with ASCE and ATC. Specific BSSC tasks were completed under the guidance of a BSSC Project Committee. To ensure project continuity and direction, a Project Oversight Committee (POC) was responsible to the BSSC Board of Direction for accomplishment of the project objectives and the conduct of project tasks. Further, a Seismic Rehabilitation Advisory Panel reviewed project products as they developed and advised the POC on the approach being taken, problems arising or anticipated, and progress made.

Three user workshops were held during the course of the project to expose the project and various drafts of the *Guidelines* documents to review by potential users of the ultimate product. The two earlier workshops provided for review of the overall project structure and for detailed review of the 50-percent-complete draft. The last workshop was held in December 1995 when the *Guidelines* documents were 75 percent complete. Participants in this workshop also had the opportunity to attend a tutorial on application of the guidelines and to comment on all project work done to date.

Following the third user workshop, written and oral comments on the 75-percent-complete draft of the documents received from the workshop participants and other reviewers were addressed by the authors and incorporated into a pre-ballot draft of the *Guidelines* and *Commentary.* POC members were sent a review copy of the 100-percent-complete draft in August 1996 and met to formulate a recommendation to the BSSC Board of Direction concerning balloting of the documents. Essentially, the POC recommended that the Board accept the documents for consensus balloting by the BSSC member organization. The Board, having received this recommendation in late August, voted unanimously to proceed with the balloting.

The balloting of the *Guidelines* and *Commentary* occurred between October 15 and December 20, 1996, and a ballot symposium for the voting representatives of BSSC member organizations was held in November during the ballot period. Member organization voting representatives were asked to vote on each major subsection of the *Guidelines* document and on each chapter of the *Commentary.* 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.

Although all sections of the *Guidelines* and *Commentary* documents were approved in the balloting, the comments and explanations received with “yes with reservations” and “no” votes were compiled by the BSSC for delivery to ATC for review and resolution. The ATC Senior Technical Committee reviewed these comments in detail and commissioned members of the technical teams to develop detailed responses and to formulate any needed proposals for change reflecting the comments. This effort resulted in 48 proposals for change to be submitted to the BSSC member organizations for a second ballot. In April 1997, the ATC presented its recommendations to the Project Oversight Committee, which approved them for forwarding to the BSSC Board. The BSSC Board subsequently gave tentative approval to the reballoting pending a mail vote on the entire second ballot package. This was done and the reballoting was officially approved by the Board. The second ballot package was mailed to BSSC member organizations on June 10 with completed ballots due by July 28.

All the second ballot proposals passed the ballot; however, as with the first ballot results, comments submitted with ballots were compiled by the BSSC for review by the ATC Senior Technical Committee. This effort resulted in a number of editorial changes and six additional technical changes being proposed by the ATC. On September 3, the ATC presented its recommendations for change to the Project Oversight Committee that, after considerable discussion, deemed the proposed changes to be either editorial or of insufficient substance to warrant another ballot. Meeting on September 4, the BSSC Board received the recommendations of the POC, accepted them, and approved preparation of the final documents for transmittal to the Federal Emergency Management Agency. This was done on September 30, 1997.

It should be noted by those using this document that recommendations resulting from the concept work of the BSSC Project Committee have resulted in initiation of a case studies project that will involve the development of seismic rehabilitation designs for at least 40 federal buildings selected from an inventory of buildings determined to be seismically deficient under the implementation program of Executive Order 12941 and determined to be considered “typical of existing structures located throughout the nation.” The case studies project is structured to:

- Test the usability of the *NEHRP Guidelines for the Seismic Rehabilitation of Buildings* in authentic applications in order to determine the extent to which practicing design engineers and architects find the *Guidelines* documents themselves and the structural analysis procedures and acceptance criteria included to be presented in understandable language and in a clear, logical fashion that permits valid engineering determinations to be made, and to evaluate the ease of transition from current engineering practices to the new concepts presented in the *Guidelines*.

- Assess the technical adequacy of the *Guidelines* design and analysis procedures. Determine if application of the procedures results (in the judgment of the designer) in rational designs of building components for corrective rehabilitation measures. Assess whether these designs adequately meet the selected performance levels when compared to existing procedures and in light of the knowledge and experience of the designer. Evaluate whether the *Guidelines* methods provide a better fundamental understanding of expected seismic performance than do existing procedures.

- Assess whether the *Guidelines* acceptance criteria are properly calibrated to result in component designs that provide permissible values of such key factors as drift, component strength demand, and inelastic deformation at selected performance levels.

- Develop empirical data on the costs of rehabilitation design and construction to meet the *Guidelines* “basic safety objective” as well as the higher performance levels included. Assess whether the anticipated higher costs of advanced engineering analysis result in worthwhile savings compared to the cost of constructing more conservative design solutions necessary with a less systematic engineering effort.

- Compare the acceptance criteria of the *Guidelines* with the prevailing seismic design requirements for new buildings in the building location to determine whether requirements for achieving the *Guidelines* "basic safety objective" are equivalent to or more or less stringent than those expected of new buildings.

Feedback from those using the *Guidelines* outside this case studies project is strongly encouraged. Further, the curriculum for a series of education/training seminars on the *Guidelines* is being developed and a number of seminars are scheduled for conduct in early 1998. Those who wish to provide feedback or with a desire for information concerning the seminars should direct their correspondence to: BSSC, 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 the *Guidelines* and *Commentary* can be obtained by phone from the FEMA Distribution Facility at 1-800-480-2520.

The BSSC Board of Direction gratefully acknowledges the contribution of all the ATC and ASCE participants in the *Guidelines* development project as well as those of the BSSC Seismic Rehabilitation Advisory Panel, the BSSC Project Committee, and the User Workshop participants. The Board also wishes to thank Ugo Morelli, FEMA Project Officer, and Diana Todd, FEMA Technical Advisor, for their valuable input and support.

Eugene Zeller
Chairman, BSSC Board of Direction

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1. Introduction

1.1 Purpose

The primary purpose of this document is to provide technically sound and nationally acceptable guidelines for the seismic rehabilitation of buildings. The *Guidelines for the Seismic Rehabilitation of Buildings* are intended to serve as a ready tool for design professionals, a reference document for building regulatory officials, and a foundation for the future development and implementation of building code provisions and standards.

This document consists of two volumes. The *Guidelines* volume details requirements and procedures, which the *Commentary* volume explains. A companion volume titled *Example Applications* contains information on typical deficiencies, rehabilitation costs, and other useful explanatory information.

This document is intended for a primary user group of architects, engineers, and building officials, specifically those in the technical community responsible for developing and using building codes and standards, and for carrying out the design and analysis of buildings. Parts of the document will also be useful and informative to such secondary audiences beyond the technical community as building owners, government agencies, and policy makers.

The engineering expertise of a design professional is a prerequisite to the appropriate use of the *Guidelines*, and most of the provisions of the following chapters presume the expertise of a professional engineer experienced in building design, as indicated in specific references to "the engineer" found extensively throughout this document.

An engineer can use this document to help a building owner select seismic protection criteria when the owner's risk reduction efforts are purely voluntary. The engineer can also use the document for the design and analysis of seismic rehabilitation projects. However, this document should not be considered to be a design manual, textbook, or handbook. Notwithstanding the instructional examples and explanations found in the *Commentary* and *Example Applications* volume, other supplementary information and instructional resources may well be required to use this document appropriately.

This document is neither a code nor a standard. It is intended to be suitable both for voluntary use by owners and design professionals as well as for adaptation and adoption into model codes and standards. Conversion of material from the *Guidelines* into a code or standard will require, as a minimum, a) careful study as to the applicability of acceptance criteria to the specific situation and building type, b) reformatting into code language, c) the addition of rules of applicability or "triggering" policies, and d) modification or addition of requirements relating to specific building department operations within a given jurisdiction.

See Section 1.3 for important descriptions of the scope and limitations of this document.

1.2 Significant New Features

This document contains several new features that depart significantly from previous seismic design procedures used to design new buildings.

1.2.1 Seismic Performance Levels and Rehabilitation Objectives

Methods and design criteria to achieve several different levels and ranges of seismic performance are defined. The four Building Performance Levels are Collapse Prevention, Life Safety, Immediate Occupancy, and Operational. (The Operational Level is defined, but specification of complete design criteria is not included in the *Guidelines*. See Chapter 2.) These levels are discrete points on a continuous scale describing the building's expected performance, or alternatively, how much damage, economic loss, and disruption may occur.

Each Building Performance Level is made up of a Structural Performance Level that describes the limiting damage state of the structural systems and a Nonstructural Performance Level that describes the limiting damage state of the nonstructural systems. Three Structural Performance Levels and four Nonstructural Performance Levels are used to form the four basic Building Performance Levels listed above.

In addition, two ranges of structural performance are defined to provide a designation for unique rehabilitations that may be intended for special purposes and therefore will fall between the rather

Building Performance Levels and Ranges

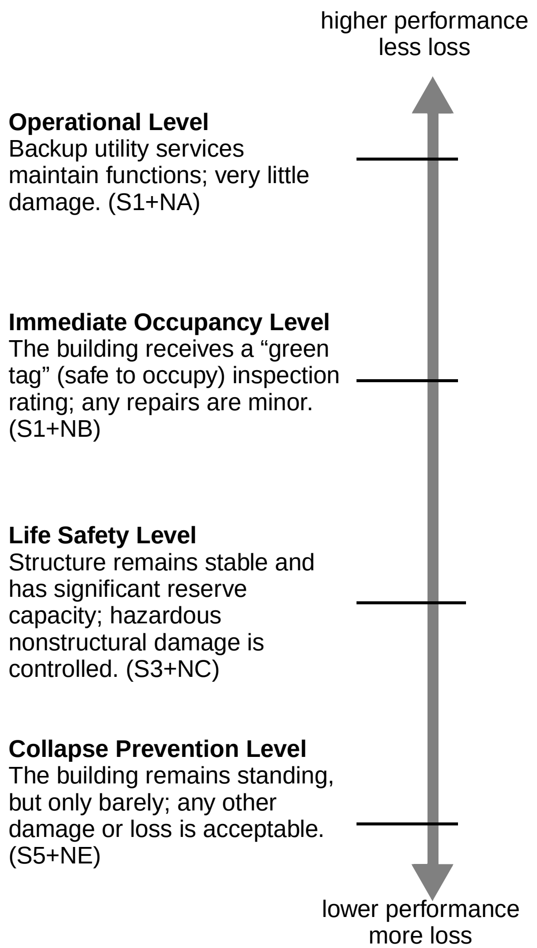
Performance Level: the intended post-earthquake condition of a building; a well-defined point on a scale measuring how much loss is caused by earthquake damage. In addition to casualties, loss may be in terms of property and operational capability.

Performance Range: a range or band of performance, rather than a discrete level.

Designations of Performance Levels and Ranges: Performance is separated into descriptions of damage of structural and nonstructural systems; structural designations are S-1 through S-5 and nonstructural designations are N-A through N-D.

Building Performance Level: The combination of a Structural Performance Level and a Nonstructural Performance Level to form a complete description of an overall damage level.

Rehabilitation Objective: The combination of a Performance Level or Range with Seismic Demand Criteria.



well-defined structural levels. Other structural and nonstructural categories are included to describe a wide range of seismic rehabilitation intentions. In fact, one of the goals of the performance level system employed in this document is to enable description of all performance objectives previously designated in codes and standards and most objectives used in voluntary rehabilitation efforts.

The three Structural Performance Levels and two Structural Performance Ranges consist of:

- S-1: Immediate Occupancy Performance Level
- S-2: Damage Control Performance Range (extends between Life Safety and Immediate Occupancy Performance Levels)
- S-3: Life Safety Performance Level
- S-4: Limited Safety Performance Range (extends between Life Safety and Collapse Prevention Performance Levels)
- S-5: Collapse Prevention Performance Level

In addition, there is the designation of S-6, Structural Performance Not Considered, to cover the situation where only nonstructural improvements are made.

The four Nonstructural Performance Levels are:

- N-A: Operational Performance Level
- N-B: Immediate Occupancy Performance Level
- N-C: Life Safety Performance Level
- N-D: Hazards Reduced Performance Level

In addition, there is the designation of N-E, Nonstructural Performance Not Considered, to cover the situation where only structural improvements are made.

A description of "what the building will look like after the earthquake" raises the questions: Which earthquake? A small one or a large one? A minor-to-moderate degree of ground shaking severity at the site where the building is located, or severe ground motion? Ground shaking criteria must be selected, along with a desired Performance Level or Range, for the *Guidelines* to be applied; this can be done either by reference to standardized regional or national ground shaking hazard maps, or by site-specific studies.

Once a desired Building Performance Level for a particular ground shaking severity (seismic demand) is selected, the result is a Rehabilitation Objective (see Section 1.5.1.3 for a detailed discussion). With the exception of the Basic Safety Objective (BSO), there are no preset combinations of performance and ground shaking hazard. The Basic Safety Objective is met when a building can satisfy two criteria: (1) the Life Safety Building Performance Level, which is the combination of the Structural and Nonstructural Life Safety Performance Levels, for the Basic Safety Earthquake 1 (BSE-1), and (2) the Collapse Prevention Performance Level, which only pertains to structural performance, for the stronger shaking that occurs less frequently as defined in the Basic Safety Earthquake 2 (BSE-2). One or more of these two levels of earthquake motion may be used in the design process to meet other Rehabilitation Objectives as well, but they have been selected as the required ground shaking criteria for the BSO. While the margin against failure may be smaller and the reliability less, the primary goal of the BSO is to provide a level of safety for rehabilitated buildings similar to that of buildings recently designed to US seismic code requirements. In fact, the strongest argument for using similar ground motions to those used for new buildings is to enable a direct comparison of expected performance. It should be remembered, however, that economic losses from damage are not explicitly considered in the BSO, and these losses in rehabilitated existing buildings should be expected to be larger than in the case of a newly constructed building.

Using various combinations of Performance Levels and ground shaking criteria, many other Rehabilitation Objectives can be defined. Those objectives that exceed the requirements for the BSO, either in terms of Performance Level, ground shaking criteria, or both, are termed Enhanced Objectives, and similarly, those that fail to meet some aspect of the BSO are termed Limited Objectives.

1.2.2 Simplified and Systematic Rehabilitation Methods

Simplified Rehabilitation may be applied to certain small buildings specified in the *Guidelines*. The primary intent of Simplified Rehabilitation is to reduce seismic risk efficiently where possible and appropriate by seeking Limited Objectives. Partial rehabilitation measures, which target high-risk building deficiencies such as parapets and other exterior falling hazards, are included as Simplified Rehabilitation techniques. Although limited in scope, Simplified Rehabilitation will be applicable to a large number of buildings throughout the US. The Simplified Rehabilitation Method employs equivalent static force analysis procedures, which are found in most seismic codes for new buildings.

Systematic Rehabilitation may be applied to any building and involves thorough checking of each existing structural element or component (an element such as a moment-resisting frame is composed of beam and column components), the design of new ones, and verification of acceptable overall interaction for expected displacements and internal forces. The Systematic Rehabilitation Method focuses on the nonlinear behavior of structural response, and employs procedures not previously emphasized in seismic codes.

1.2.3 Varying Methods of Analysis

Four distinct analytical procedures can be used in Systematic Rehabilitation: Linear Static, Linear Dynamic, Nonlinear Static, and Nonlinear Dynamic Procedures. The choice of analytical method is subject to limitations based on building characteristics. The linear procedures maintain the traditional use of a linear stress-strain relationship, but incorporate adjustments to overall building deformations and material acceptance criteria to permit better consideration of the probable nonlinear characteristics of seismic response. The Nonlinear Static Procedure, often called “pushover analysis,” uses simplified nonlinear techniques to estimate seismic structural deformations. The Nonlinear Dynamic Procedure, commonly known as nonlinear time history analysis, requires considerable judgment and experience to perform, and may only be used within the limitations described in Section 2.9.2.2 of the *Guidelines*.

1.2.4 Quantitative Specifications of Component Behavior

Inherent in the concept of Performance Levels and Ranges is the assumption that performance can be measured using analytical results such as story drift ratios or strength and ductility demands on individual components or elements. To enable structural verification at the selected Performance Level, stiffness, strength, and ductility characteristics of many common elements and components have been derived from laboratory tests and analytical studies and put in a standard format in the *Guidelines*.

1.2.5 Procedures for Incorporating New Information and Technologies into Rehabilitation

It is expected that testing of existing materials and elements will continue and that additional corrective measures and products will be developed. It is also expected that systems and products intended to modify structural response beneficially will be advanced. The format of the analysis techniques and acceptability criteria of the *Guidelines* allows rapid incorporation of such technology. Section 2.13 gives specific guidance in this regard. It is expected that the *Guidelines* will have a significant impact on testing and documentation of existing materials and systems as well as new products. In addition, an entire chapter (Chapter 9) has been devoted to two such new technologies, seismic isolation and energy dissipation.

1.3 Scope, Contents, and Limitations

This section describes the scope and limitations of the contents of this document pertaining to the following:

- buildings and loadings
- activities and policies associated with seismic rehabilitation
- seismic mapping
- technical content

1.3.1 Buildings and Loadings

This document is intended to be applied to all buildings—regardless of importance, occupancy, historic features, size, or other characteristics—that by some criteria are deficient in their ability to resist the effects of earthquakes. In addition to the direct effects of ground shaking, this document also considers the effects on buildings of local ground failure such as liquefaction. With careful extrapolation, the procedures herein can also be applied to many nonbuilding structures such as pipe racks, steel storage racks, structural towers for tanks and vessels, piers, wharves, and electrical power generating facilities. The applicability of the procedures has not been examined for each and every structural type, particularly those that have generally been covered by their own codes or standards, such as bridges and nuclear power plants. It is important to note that, as written, the provisions are not intended to be mandatory. Careful consideration of the applicability to any given group of buildings or structures should be made prior to adoption of any portion of these procedures for mandatory use.

This document applies to the seismic resistance of both the overall structural system of a building and its elements—such as shear walls or frames—and the constituent components of elements, such as a column in a frame or a boundary member in a wall. It also applies to nonstructural components of existing buildings—ceilings, partitions, and mechanical/ electrical systems. In addition to techniques for increasing strength and ductility of systems, this document provides rehabilitation techniques for reducing seismic demand, such as the introduction of isolation or damping devices. And, although this document is not intended to address the design of new buildings, it does cover new components or elements to be added to existing buildings. Evaluation of components for gravity and wind forces in the absence of earthquake demands is beyond the scope of the document.

1.3.2 Activities and Policies Associated with Seismic Rehabilitation

There are several significant steps in the process of reducing seismic risk in buildings that this document does not encompass. The first step, deciding whether or not to undertake a rehabilitation project for a particular building, is beyond the scope of the *Guidelines*. Once the decision to rehabilitate a building has been made, the *Guidelines'* detailed engineering guidance on how to conduct seismic rehabilitation analysis can be applied.

Another step, determining when the *Guidelines* should be applicable in a mandatory way to a remodeling or structural alteration project (the decision as to when the provisions are “triggered”), is also beyond the scope of this document. Finally, methods of reducing seismic risk that do not physically change the building—such as reducing the number of occupants—are not covered here.

Recommendations regarding the selection of a Rehabilitation Objective for any building are also beyond the scope of this document. As noted above, a life safety risk often considered acceptable, is defined by a specific objective, termed the Basic Safety Objective (BSO). Higher and lower objectives can also be defined by the user. The *Commentary* discusses issues to consider when combining various performance and seismic hazard levels; it should be noted that not all combinations constitute reasonable or cost-effective Rehabilitation Objectives. The *Guidelines* were written under the premise that greater flexibility is required in seismic rehabilitation than in the design of new buildings. However, even with the flexibility provided by various Rehabilitation Objectives, once a Rehabilitation Objective is decided upon, the *Guidelines* provide internally consistent procedures that include the necessary analysis and construction specifications.

Featured in the *Guidelines* are descriptions of damage states with relation to specific Performance Levels. These descriptions are intended to aid design professionals and owners when selecting appropriate Performance Levels for rehabilitation design. They are not intended to be used directly for condition assessment of earthquake-damaged buildings. Although there are similarities in damage descriptions that are used for selection of rehabilitation design criteria and descriptions used for post-earthquake damage assessment, many factors enter into the design and assessment processes. No single parameter should be cited as defining either a Performance Level or the safety or usefulness of an earthquake-damaged building.

Techniques of repair for earthquake-damaged buildings are not included in the *Guidelines*. However, if the mechanical properties of repaired components are known, acceptability criteria for use in this document can be either deduced by comparison with other similar components, or derived. Any combination of repaired elements, undamaged existing elements, and new elements can be modeled using this document, and each checked against Performance Level acceptance criteria.

Although the *Guidelines* were not written for the purpose of evaluating the expected performance of an unrehabilitated existing building, they may be used as a reference for evaluation purposes in deciding whether a building requires rehabilitation, similarly to the way code provisions for new buildings are sometimes used as an evaluation tool.

1.3.3 Seismic Mapping

Special or new mapping of expected seismic ground shaking for the country has not been developed for the *Guidelines*. However, new national earthquake hazard maps were developed in 1996 by the United States Geological Survey (USGS) as part of a joint project (known as Project ’97) with the Building Seismic Safety Council to update the 1997 *NEHRP Recommended Provisions* for new buildings. National probabilistic maps were developed for ground motions with a 10% chance of exceedance in 50 years, a 10% chance of exceedance in 100 years (which can also be expressed as a 5% chance of exceedance in 50 years) and a 10% chance of exceedance in 250 years (which also can be expressed as a 2% chance of exceedance in 50 years). These probabilities correspond to motions that are expected to occur, on average, about once every 500, 1000, and 2500 years. In addition, in certain locations with well-defined earthquake sources, local ground motions for specific earthquakes were developed, known as deterministic motions. Key ordinates of a ground motion response spectrum for these various cases allow the user to develop a complete spectrum at any site. The *Guidelines* are written to use such a response spectrum as the seismic demand input for the various analysis techniques.

The responsibility of the Building Seismic Safety Council in Project ’97 was to develop a national map and/or analytical procedure to best utilize the new seismic hazard information for the design of new buildings. As part of that process, rules were developed to combine portions of both the USGS probabilistic and deterministic maps to create a map of ground motions representing the effects of large, rare events in all parts of the country. This event is called the Maximum Considered Earthquake (MCE). New buildings are to be designed, with traditional design rules, for two-thirds of these ground motion values with the purpose of providing an equal margin against collapse for the varied seismicity across the country.

For consistency in this document, ground motion probabilities will be expressed with relationship to 50-year exposure times, and in a shorthand format; i.e., 10%/50 years is a 10% chance of exceedance in 50 years, 5%/50 years is a 5% chance of exceedance in 50 years, and 2%/50 years is a 2% chance of exceedance in 50 years.

The variable Rehabilitation Objectives featured in the *Guidelines* allows consideration of any ground motion that may be of interest, the characteristics of which can be determined specifically for the site, or taken from a national or local map. However, specifically for use with the BSO, and generally for convenience in defining the ground motion for other Rehabilitation Objectives, the 10%/50 year probabilistic maps and the MCE maps developed in Project 97 are in the map package distributed with the *Guidelines*. For additional map packages, call FEMA at 1-800-480-2520.

New ground motion maps specifically related to the seismic design procedures of the 1997 *NEHRP Recommended Provisions* are expected to be available. These maps plot key ordinates of a ground motion response spectrum, allowing development by the user of a complete spectrum at any site. The *Guidelines* are written to use such a response spectrum as the seismic demand input for the various analysis techniques. While the NEHRP maps provide a ready source for this type of information, the *Guidelines* may be used with seismic hazard data from any source as long as it is expressed as a response spectrum.

1.3.4 Technical Content

The *Guidelines* have been developed by a large team of specialists in earthquake engineering and seismic rehabilitation. The most advanced analytical techniques that were considered practical for production use have been incorporated, and seismic Performance Level criteria have been specified using actual laboratory test results, where available, supplemented by the engineering judgment of the various development teams. Certain buildings damaged in the 1994 Northridge earthquake and a limited number of designs using codes for new buildings have been checked with the procedures of this document. There has not yet been the opportunity, however, for comprehensive comparisons with other codes and standards, nor for evaluation of the accuracy in predicting the damage level under actual earthquake ground motions. As of this writing (1997), significant case studies are already underway to test more thoroughly the various analysis techniques and acceptability criteria. There undoubtedly will also be lessons learned from future damaging earthquakes by studying performance of both unrehabilitated buildings and buildings rehabilitated to these or other standards. A structured program will also be instituted to gather and assess the new knowledge relevant to the data, procedures, and criteria contained in the *Guidelines*, and make recommendations for future refinements. Engineering judgment should be exercised in determining the applicability of various analysis techniques and material acceptability criteria in each situation. It is suggested that results obtained for any individual building be validated by additional checks using alternative methodologies and careful analysis of any differences. Information contained in the *Commentary* will be valuable for such individual validation studies.

The concepts and terminology of performance-based design are new and should be carefully studied and discussed with building owners before use. The terminology used for Performance Levels is intended to represent *goals* of design. The actual ground motion will seldom be comparable to that specified in the Rehabilitation Objective, so in most events, designs targeted at various damage states may only determine relative performance. Even given a ground motion similar to that specified in the Rehabilitation Objective and used in design, variations from stated performances should be expected. These could be associated with unknown geometry and member sizes in existing buildings, deterioration of materials, incomplete site data, variation of ground motion that can occur within a small area, and incomplete knowledge and simplifications related to modeling and analysis. Compliance with the *Guidelines* should therefore not be considered a guarantee of the specified performance. Determination of statistical reliability of the recommendations in the *Guidelines* was not a part of the development project. Such a study would require development of and consensus acceptance of a new methodology to determine reliability. However, the expected reliability of achieving various Performance Levels when the requirements of a given Level are followed is discussed in the *Commentary* for Chapter 2.

1.4 Relationship to Other Documents and Procedures

The *Guidelines* contain specific references to many other documents; however, the *Guidelines* are also related generically to the following publications.

- FEMA 222A and 223A, *NEHRP Recommended Provisions for Seismic Regulations for New Buildings* (BSSC, 1995): For the purposes of the design of new components, the *Guidelines* have been designed to be as compatible as possible with the companion *Provisions* for new buildings and its reference design documents. Detailed references to the use of specific sections of the *Provisions* document will be found in subsequent sections of the *Guidelines*.

- FEMA 302 and 303, 1997 *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* (BSSC, 1997), referred to herein as the 1997 *NEHRP Recommended Provisions*, have been in preparation for the same time as the later versions of the *Guidelines*. Most references are to the 1994 *NEHRP Recommended Provisions.*

- FEMA 237, *Development of Guidelines for Seismic Rehabilitation of Buildings, Phase I: Issues Identification and Resolution* (ATC, 1992), which underwent an American Society of Civil Engineers (ASCE) consensus approval process, provided policy direction for this document.

- *Proceedings of the Workshop To Resolve Seismic Rehabilitation Sub-issues* (ATC, 1993) provided recommendations to the writers of the *Guidelines* on more detailed sub-issues.

- FEMA 172, *NEHRP Handbook of Techniques for the Seismic Rehabilitation of Existing Buildings* (BSSC, 1992a), originally produced by URS/Blume and reviewed by the BSSC, contains construction techniques for implementing engineering solutions to the seismic deficiencies of existing buildings.

- FEMA 178, *NEHRP Handbook for the Seismic Evaluation of Existing Buildings* (BSSC, 1992b), which was originally developed by ATC and underwent the consensus approval process of the BSSC, covers the subject of evaluating existing buildings to decide if they are seismically deficient in terms of life safety. The model building types and other information from that publication are used or referred to extensively in the *Guidelines* in Chapter 10 and in the *Example Applications* document (ATC, 1997). FEMA 178, 1992 edition, is being updated to include additional performance objectives as well as to be more compatible with the *Guidelines*.

- FEMA 156 and 157, Second Edition, *Typical Costs for Seismic Rehabilitation of Existing Buildings* (Hart, 1994 and 1995), reports statistical analysis of the costs of rehabilitation of over 2000 buildings, based on construction costs or detailed studies. Several different seismic zones and performance levels are included in the data. Since the data were developed in 1994, none of the data is based on buildings rehabilitated specifically in accordance with the current *Guidelines* document. Performance Levels defined in the *Guidelines* are not intended to be significantly different from parallel levels used previously, and costs should still be reasonably representative.

- FEMA 275, *Planning for Seismic Rehabilitation: Societal Issues (VSP, 1996),* discusses societal and implementation issues associated with rehabilitation, and describes several case histories.

- FEMA 276, *Guidelines for the Seismic Rehabilitation of Buildings*: *Example Applications* (ATC, 1997), intended as a companion document to the *Guidelines* and *Commentary,* describes examples of buildings that have been seismically rehabilitated in various seismic regions and for different Rehabilitation Objectives. Costs of the work are given and references made to FEMA 156 and 157. Since the document is based on previous case histories, none of the examples were rehabilitated specifically in accordance with the current Guidelines document. However, Performance Levels defined in the *Guidelines* are not intended to be significantly different than parallel levels used previously, and the case studies are therefore considered representative.

- ATC 40, *Seismic Evaluation and Retrofit of Concrete Buildings*, (ATC, 1996), incorporates performance levels almost identical to those shown in Table 2-9 and employs “pushover” nonlinear analysis techniques. The capacity spectrum method for determining the displacement demand is treated in detail. This document covers only concrete buildings.

1.5 Use of the *Guidelines* in the Seismic Rehabilitation Process

Figure 1-1 is an overview of the flow of procedures contained in this document as well as an indication of the broader scope of the overall seismic rehabilitation process for individual buildings. In addition to showing a simplified flow diagram of the overall process, Figure 1-1 indicates points at which input from this document is likely, as well as potential steps outside the scope of the *Guidelines*. Specific chapter references are noted at points in the flow diagram where input from the *Guidelines* is to be obtained. This is a very general depiction of this process, which can take many forms and may include steps more numerous and in different order than shown.

As indicated in Section 1.3, the *Guidelines* are written with the assumption that the user has already concluded that a building needs to be seismically improved; evaluation techniques for reaching this decision are not specifically prescribed. However, the use of the detailed analysis and verification techniques associated with Systematic Rehabilitation (Section 1.5.4) may indicate that some buildings determined to be deficient by other evaluation or classification systems are actually acceptable without modification. This might occur, for example, if a *Guidelines* analysis method reveals that an existing building has greater capacity than was determined by use of a less exact evaluation method.

1.5.1 Initial Considerations for Individual Buildings

The use of the *Guidelines* will be simplified and made more efficient if certain base information is obtained and considered prior to beginning the process.

The building owner should be aware of the range of costs and impacts of rehabilitation, including both the variation associated with different Rehabilitation Objectives and the potential add-on costs often associated with seismic rehabilitation, such as other life safety upgrades, hazardous material removal, work associated with the Americans with Disabilities Act, and nonseismic building remodeling. Also to be considered are potential federal tax incentives for the rehabilitation for historic buildings and for some other older nonresidential buildings.

The use of the building must be considered in weighing the significance of potential temporary or permanent disruptions associated with various risk mitigation schemes. Other limitations on modifications to the building due to historic or aesthetic features must also be understood. The historic status of every building at least 50 years old should be determined (see the sidebar, *Considerations for Historic Buildings,* later in this chapter). This determination should be made early, because it could influence the choices of rehabilitation approaches and techniques.

This document is focused primarily on the technical aspects of rehabilitation. Basic information specifically included in the *Guidelines* is discussed below.

1.5.1.1 Site Hazards Other than Seismic Ground Shaking

The analysis and design procedures of the *Guidelines* are primarily aimed at improving the performance of buildings under the loads and deformations imposed by seismic shaking. However, other seismic hazards could exist at the building site that could damage the building regardless of its ability to resist ground shaking. These hazards include fault rupture, liquefaction or other shaking-induced soil failures, landslides, and inundation from offsite effects such as dam failure or tsunami.

The risk and possible extent of damage from such site hazards should be considered before undertaking rehabilitation aimed solely at reducing shaking damage. In some situations, it may be feasible to mitigate the site hazard. In many cases, the likelihood of the site hazard occurring will be sufficiently small that rehabilitating the building for shaking alone is appropriate. Where a site hazard exists, it may be feasible to mitigate it, either by itself or in connection with the building rehabilitation project. It is also possible that the risk from a site hazard is so extreme and difficult to control that rehabilitation will not be cost-effective.

Chapter 2 describes the applicability of seismic ground failure hazards to this document's seismic rehabilitation requirements, and Chapter 4 describes corresponding analysis procedures and mitigation measures.

1.5.1.2 Characteristics of the Existing Building

Chapter 2 discusses investigation of as-built conditions. Efficient use of the *Guidelines* requires basic knowledge of the configuration, structural characteristics, and seismic deficiencies of the building. Much of this information will normally be available from a seismic evaluation of the building. For situations where seismic rehabilitation has been mandated by local government according to building construction classification, familiarity with the building type and its typical seismic deficiencies is recommended. Such information is available from several sources, including FEMA 178 (BSSC, 1992b) and the companion *Example Applications* document.

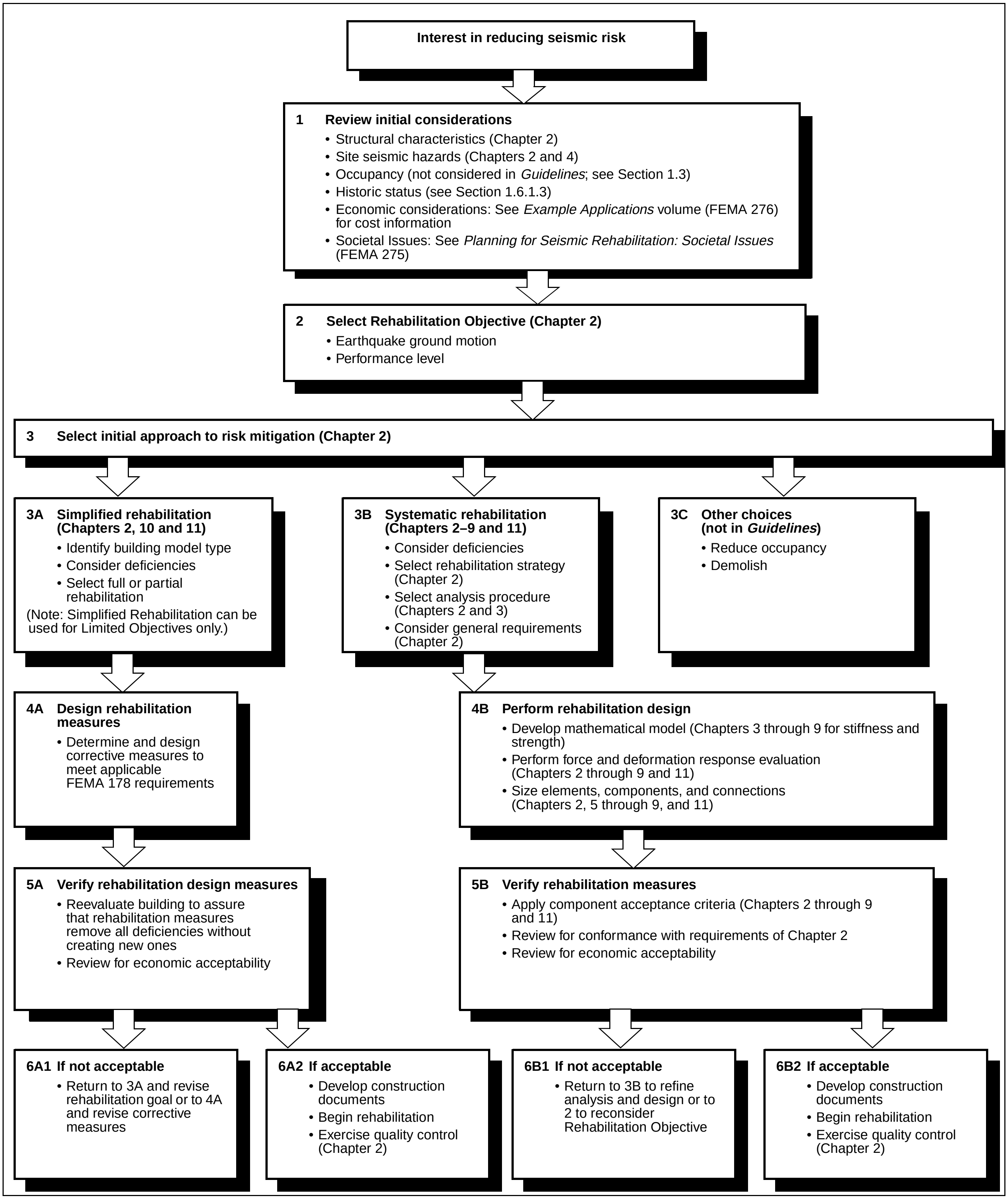


Figure 1-1 Rehabilitation Process Flowchart

Basic information about the building is needed to determine eligibility for Simplified Rehabilitation (Step 3 in Figure 1-1), if its use is desired, or to develop a preliminary design (Step 4 in Figure 1-1). It is prudent to perform preliminary calculations to select key locations or parameters prior to establishing a detailed testing program, in order to obtain knowledge cost-effectively and with as little disruption as possible of construction features and materials properties at concealed locations.

If the building is historic, additional as-built conditions should be more thoroughly investigated and analyzed. Publications dealing with the specialized subject of the character-defining spaces, features, and details of historic buildings should be consulted, and the services of a historic preservation expert may be required.

1.5.1.3 Rehabilitation Objective

A Rehabilitation Objective must be selected, at least on a preliminary basis, before beginning to use the procedures of the *Guidelines*. A Rehabilitation Objective is a statement of the desired limits of damage or loss (Performance Level) for a given seismic demand. The selection of a Rehabilitation Objective will be made by the owner and engineer in voluntary rehabilitation cases, or by relevant public agencies in mandatory programs. If the building is historic, there should be an additional goal to preserve its historic fabric and character in conformance with the Secretary of the Interior's *Standards for Rehabilitation*.

Whenever possible, the Rehabilitation Objective should meet the requirements of the BSO, which consists of two parts: 1, the Life Safety Building Performance Level for BSE-1 (the earthquake ground motion with a 10% chance of exceedance in 50 years (10%/50 year), but in no case exceeding two-thirds of the ground response expressed for the Maximum Considered Earthquake) and 2, the Collapse Prevention Building Performance Level for the earthquake ground motion representing the large, rare event, called the Maximum Considered Earthquake (described in the *Guidelines* as BSE-2). Throughout this document, the BSO provides a national benchmark with which lower or higher Rehabilitation Objectives can be compared.

Due to the variation in performance associated with unknown conditions in existing buildings, deterioration of materials, incomplete site data, and large variation expected in ground shaking, compliance with the *Guidelines* should not be considered a guarantee of the specified performance. The expected reliability of achieving various Performance Levels when the requirements of a given Level are followed is discussed in the *Commentary* to Chapter 2.

1.5.2 Initial Risk Mitigation Strategies

There are many ways to reduce seismic risk, whether the risk is to property, life safety, or post-earthquake use of the building. The occupancy of vulnerable buildings can be reduced, redundant facilities can be provided, and nonhistoric buildings can be demolished and replaced. The risks posed by nonstructural components and contents can be reduced. Seismic site hazards other than shaking can be mitigated.

Most often, however, when all alternatives are considered, the options of modifying the building to reduce the risk of damage must be studied. Such corrective measures include stiffening or strengthening the structure, adding local elements to eliminate irregularities or tie the structure together, reducing the demand on the structure through the use of seismic isolation or energy dissipation devices, and reducing the height or mass of the structure. These modification strategies are discussed in Chapter 2.

Modifications appropriate to the building can be determined using either the Simplified Rehabilitation Method or Systematic Rehabilitation Method.

1.5.3 Simplified Rehabilitation

Simplified Rehabilitation will apply to many small buildings of regular configuration, particularly in moderate or low seismic zones. Simplified Rehabilitation requires less complicated analysis and in some cases less design than the complete analytical rehabilitation design procedures found under Systematic Rehabilitation. In many cases, Simplified Rehabilitation represents a cost-effective improvement in seismic performance, but often does not require sufficiently detailed or complete analysis and evaluation to qualify for a specific Performance Level. Simplified Rehabilitation techniques are described for components (e.g., parapets, wall ties), as well as entire systems. Simplified Rehabilitation of structural systems is covered in Chapter 10, and the combinations of seismicity, Model Building, and other considerations for which it is allowed are provided in Section 2.8 and in Table 10-1. Simplified rehabilitation of nonstructural components is covered in Chapter 11.

1.5.4 Systematic Rehabilitation

The Systematic Rehabilitation Method is intended to be complete and contains all requirements to reach any specified Performance Level. Systematic Rehabilitation is an iterative process, similar to the design of new buildings, in which modifications of the existing structure are assumed for the purposes of a preliminary design and analysis, and the results of the analysis are verified as acceptable on an element and component basis. If either new or existing components or elements still prove to be inadequate, the modifications are adjusted and, if necessary, a new analysis and verification cycle is performed. Systematic Rehabilitation is covered in Chapters 2 through 9, and 11.

1.5.4.1 Preliminary Design

A preliminary design is needed to define the extent and configuration of corrective measures in sufficient detail to estimate the interaction of the stiffness, strength, and post-yield behavior of all new, modified, or existing elements to be used for lateral force resistance. The designer is encouraged to include all elements with significant lateral stiffness in a mathematical model to assure deformation capability under realistic seismic drifts. However, just as in the design of new buildings, it may be determined that certain components or elements will not be considered part of the lateral-force-resisting system, as long as deformation compatibility checks are made on these components or elements to assure their adequacy. In Figure 1-1, the preliminary design is in Steps 3 and 4.

1.5.4.2 Analysis

A mathematical model, developed for the preliminary design, must be constructed in connection with one of the analysis procedures defined in Chapter 3. These are the linear procedures (Linear Static and Linear Dynamic) and the nonlinear procedures (Nonlinear Static and Nonlinear Dynamic). With the exception of the Nonlinear Dynamic Procedure, the *Guidelines* define the analysis and rehabilitation design procedures sufficiently that compliance can be checked by a building department in a manner similar to design reviews for new buildings. Modeling assumptions to be used in various situations are given in Chapters 4 through 9, and Chapter 11 for nonstructural components, and guidance on required seismic demand is given in Chapter 2. Guidance is given for the use of the Nonlinear Dynamic Procedure; however, considerable judgment is required in its application. Criteria for applying ground motion for various analysis procedures is given, but definitive rules for developing ground motion input are not included in the *Guidelines.*

1.5.5 Verification and Economic Acceptance

For systematic rehabilitation, the effects of forces and displacements imposed on various elements by the seismic demand must be checked for acceptability for the selected Performance Level. These acceptability criteria, generally categorized by material, are given in Chapters 4 through 9. In addition, certain overall detailing, configuration, and connectivity requirements, covered in Chapter 2 and in Chapter 10 for simplified rehabilitation, must be satisfied prior to complete acceptance of the rehabilitation design. Nonstructural components are covered in Chapter 11. At this stage a cost estimate can be made to review the design's economic acceptability.

If the design proves uneconomical or otherwise unfeasible, different Rehabilitation Objectives or risk mitigation strategies may have to be considered, and the process would begin anew at Step 2 or 3 in Figure 1-1. The process would return to Step 3 or 4 if only refinements were needed in the design, or if a different scheme were to be tested.

1.5.6 Implementation of the Design

When a satisfactory design is completed, the important implementation phase may begin. Chapter 2 contains provisions for a quality assurance program during construction. While detailed analysis of construction costs and scheduling is not covered by the procedures in the *Guidelines*, these important issues are discussed in the *Example Applications* volume (ATC, 1997). Other significant aspects of the implementation process—including details of the preparation of construction documents by the architectural and engineering design professionals, obtaining a building permit, selection of a contractor, details of historic preservation techniques for particular kinds of materials, and financing—are not part of the *Guidelines*.

Social, Economic, and Political Considerations

The scope of the *Guidelines* is limited to the engineering basis for seismically rehabilitating a building, but the user should also be aware of significant nonengineering issues and social and economic impacts. These problems and opportunities, which vary with each situation, are discussed in a separate publication, *Planning for Seismic Rehabilitation: Societal Issues* (FEMA 275).

Construction Cost
If seismic rehabilitation were always inexpensive, the social and political costs and controversies would largely disappear. Unfortunately, seismic rehabilitation often requires removal of architectural materials to access the vulnerable portions of the structure, and nonseismic upgrading (e.g., electrical, handicapped access, historic restoration) is frequently "triggered" by a building code's remodeling permit requirements or is desirable to undertake at the same time.

Housing
While seismic rehabilitation ultimately improves the housing stock, units can be temporarily lost during the construction phase, which may last more than a year. This can require relocation of tenants.

Impacts on Lower-Income Groups
Lower-income residents and commercial tenants can be displaced by seismic rehabilitation. Often caused by upgrading unrelated to earthquake concerns, seismic upgrading also tends to raise rents and real estate prices, because of the need to recover the costs of the investment.

Regulations
As with efforts to impose safety regulations in other fields, mandating seismic rehabilitation is often controversial. The *Guidelines* are not written as mandatory code provisions, but one possible application is to adapt them for that use. In such cases political controversy should be expected, and nonengineering issues of all kinds should be carefully considered.

Architecture
Even if a building is not historic, there are often significant architectural impacts. The exterior and interior appearance may change, and the division of spaces and arrangement of circulation routes may be altered.

Community Revitalization
Seismic rehabilitation not only poses issues and implies costs, it also confers benefits. In addition to enhanced public safety and economic protection from earthquake loss, seismic rehabilitation can play a leading role in the revitalization of older commercial and industrial areas as well as residential neighborhoods.

1.6 Use of the *Guidelines* for Local or Directed Risk Mitigation Programs

The *Guidelines* have been written to accommodate use in a wide variety of situations, including both local risk mitigation programs and directed programs created by broadly based organizations or governmental agencies that have jurisdiction over many buildings. These programs may target certain building types for rehabilitation or require complete rehabilitation coupled with other remodeling work. The incorporation of variable Rehabilitation Objectives and use of Model Building Types in the *Guidelines* allows creation of subsets of rehabilitation requirements to suit local conditions of seismicity, building inventory, social and economic considerations, and other factors. Provisions appropriate for local situations can be extracted, put into regulatory language, and adopted into appropriate codes, standards, or local ordinances.

1.6.1 Initial Considerations for Mitigation Programs

Local or directed programs can either target high-risk building types or set overall priorities. These decisions should be made with full consideration of physical, social, historic, and economic characteristics of the building inventory. Although financial incentives can induce voluntary risk mitigation, carefully planned mandatory or directed programs, developed in cooperation with those whose interests are affected, are generally more effective. Potential benefits of such programs include reduction of direct earthquake losses—such as casualties, costs to repair damage, and loss of use of buildings—as well as more rapid overall recovery. Rehabilitated buildings may also increase in value and be assigned lower insurance rates. Additional issues that should be considered for positive or negative effects include the interaction of rehabilitation with overall planning goals, historic preservation, and the local economy. These issues are discussed in *Planning for Seismic Rehabilitation: Societal Issues* (VSP, 1996).

1.6.1.1 Potential Costs of Local or Directed Programs

The primary costs of seismic rehabilitation—the construction work itself, including design, inspection, and administration—are normally paid by the owner. Additional costs that should be weighed when creating seismic risk reduction programs are those associated with developing and administering the program, such as the costs of identification of high-risk buildings, environmental or socioeconomic impact reports, training programs, plan checking and construction inspection.

The construction costs include not only the cost of the pure structural rehabilitation but also the costs associated with new or replaced finishes that may be required. In some cases, seismic rehabilitation work will trigger other local jurisdictional requirements, such as hazardous material removal or partial or full compliance with the Americans with Disabilities Act. The costs of seismic or functional improvements to nonstructural systems should also be considered. There may also be costs to the owner associated with temporary disruption or loss of use of the building during construction. To offset these costs, there may be low-interest earthquake rehabilitation loans available from state or local government, or historic building tax credits.

If seismic rehabilitation is the primary purpose of construction, the costs of the various nonseismic work that may be required should be included as direct consequences. On the other hand, if the seismic work is an added feature of a major remodel, the nonseismic improvements probably would have been required anyway, and therefore should not be attributed to seismic rehabilitation.

A discussion of these issues, as well as guidance on the range of costs of seismic rehabilitation, is included in FEMA 156 and 157, Second Edition, *Typical Costs for Seismic Rehabilitation of Buildings* (Hart, 1994 and 1995) and in FEMA 276, *Guidelines for the Seismic Rehabilitation of Buildings: Example Applications* (ATC, 1997). Since the data for these documents were developed prior to the Guidelines, the information is not based on buildings rehabilitated specifically in accordance with the current document. However, Performance Levels defined in the *Guidelines* are not intended to be significantly different than parallel levels used previously, and costs should still be reasonably representative.

1.6.1.2 Timetables and Effectiveness

Presuming that new buildings are being constructed with adequate seismic protection and that older buildings are occasionally demolished or replaced, the inventory of seismically hazardous buildings in any community will be gradually reduced. This attrition rate is normally small, since the structures of many buildings have useful lives of 100 years or more and very few buildings are actually demolished. If buildings or districts become historically significant, they may not be subject to attrition at all. In many cases, then, doing nothing (or waiting for an outside influence to force action) may present a large cumulative risk to the inventory.

It has often been pointed out that exposure time is a significant element of risk. The time aspect of risk reduction is so compelling that it often appears as part of book and workshop titles; for example, *Between Two Earthquakes: Cultural Property in Seismic Zones* (Feilden, 1987); *Competing Against Time* (California Governor’s Board of Inquiry, 1990); and “In Wait for the Next One” (EERI, 1995). Therefore, an important consideration in the development of programs is the time allotted to reach a certain risk reduction goal. It is generally assumed that longer programs create less hardship than short ones by allowing more flexibility in planning for the cost and possible disruption of rehabilitation, as well as by allowing natural or accelerated attrition to reduce undesirable impacts. On the other hand, the net reduction of risk is smaller due to the increased exposure time of the seismically deficient building stock.

Given a high perceived danger and certain advantageous characteristics of ownership, size, and occupancy of the target buildings, mandatory programs have been completed in as little as five to ten years. More extensive programs—involving complex buildings such as hospitals, or with significant funding limitations—may have completion goals of 30 to 50 years. Deadlines for individual buildings are also often determined by the risk presented by building type, occupancy, location, soil type, funding availability, or other factors.

1.6.1.3 Historic Preservation

Seismic rehabilitation of buildings can affect historic preservation in two ways. First, the introduction of new elements that will be associated with the rehabilitation may in some way impact the historic fabric of the

Considerations for Historic Buildings

It must be determined early in the process whether a building is "historic." A building is historic if it is at least 50 years old and is listed in or potentially eligible for the National Register of Historic Places and/or a state or local register as an individual structure or as a contributing structure in a district. Structures less than 50 years old may also be historic if they possess exceptional significance. For historic buildings, users should develop and evaluate alternative solutions with regard to their effect on the loss of historic character and fabric, using the Secretary of the Interior's *Standards for Rehabilitation* (Secretary of the Interior, 1990).

In addition to rehabilitation, the Secretary of the Interior also has standards for preservation, restoration, and reconstruction. These are published in the *Standards for the Treatment of Historic Properties* (Secretary of the Interior, 1992). A seismic rehabilitation project may include work that falls under the *Rehabilitation Standards,* the *Treatment Standards*, or both.

For historic buildings as well as for other structures of architectural interest, it is important to note that the Secretary of the Interior's *Standards* define rehabilitation as "the process of returning a property to a state of utility, through repair or alteration, which makes possible an efficient contemporary use while preserving those portions and features of the property which are significant to its historic, architectural and cultural values." The Secretary has also published standards for "preservation," "restoration," and "reconstruction." Further guidance on the treatment of historic properties is contained in the publications in the *Catalog of Historic Preservation Publications* (NPS, 1995).

Rehabilitation Objectives

If seismic rehabilitation is required by the governing building jurisdiction, the minimum seismic requirements should be matched with a Rehabilitation Objective defined in the *Guidelines*. It should be noted that many codes covering historic buildings allow some amount of flexibility in required performance, depending on the effect of rehabilitation on important historic features.

If a building contains items of unusual architectural interest, consideration should be given to the value of these items. It may be desirable to rehabilitate the building to the Damage Control Performance Range to ensure that the architectural fabric survives certain earthquakes.

Rehabilitation Strategies

In development of initial risk mitigation strategies, consideration must be given to the architectural and historic value of the building and its fabric. Development of a Historic Structure Report identifying the primary historic fabric may be essential in the preliminary planning stages for certain buildings. Some structurally adequate solutions may nevertheless be unacceptable because they involve destruction of historic fabric or character. Alternate rehabilitation methods that lessen the impact on the historic fabric should be developed for consideration. Partial demolition may be inappropriate for historic structures. Elements that create irregularities may be essential to the historic character of the structure. The advice of historic preservation experts may be necessary.

Structural rehabilitation of historic buildings may be accomplished by hiding the new structural members or by exposing them as admittedly new elements in the building's history. Often, the exposure of new structural members is preferred, because alterations of this kind are "reversible"; that is, they could conceivably be undone at a future time with no loss of historic fabric to the building. The decision to hide or expose structural members is a complex one, best made by a preservation professional.

building. Second, the seismic rehabilitation work can serve to better protect the building from possibly unrepairable future earthquake damage. The effects of any seismic risk reduction program on historic buildings or preservation districts should be carefully considered during program development, and subsequent work should be carefully monitored to assure compliance with previously mentioned national preservation guidelines. (See the sidebar, "Considerations for Historic Buildings.")

1.6.2 Use in Passive Programs

Programs that only require seismic rehabilitation in association with other activity on the building are often classified as "passive." "Active" programs, on the other hand, are those that mandate seismic rehabilitation for targeted buildings in a certain time frame, regardless of other activity associated with the building (see Section 1.6.3). Activities in a building that may passively generate a requirement to seismically rehabilitate—such as an increase in occupancy, structural modification, or a major remodeling that would significantly extend the life of the building—are called "triggers." The concept of certain activities triggering compliance with current standards is well established in building codes. However, the details of the requirements have varied widely. These issues have been documented with respect to seismic rehabilitation in California (Hoover, 1992). Passive programs reduce risk more slowly than active programs.

1.6.2.1 Selection of Seismic Rehabilitation Triggers

The *Guidelines* do not cover triggers for seismic rehabilitation. The extent and detail of seismic triggers will greatly affect the speed, effectiveness, and impacts of seismic risk reduction, and the selection of triggers is a policy decision expected to be done locally, by the person or agency responsible for the inventory. Triggers that have been used or considered in the past include revision of specified proportions of the structure, remodeling of specified percentages of the building area, work on the building that costs over a specified percentage of the building value, change in use that increases the occupancy or importance of the building, and changes of ownership.

1.6.2.2 Selection of Passive Seismic Rehabilitation Standards

The *Guidelines* purposely afford a wide variety of options that can be adopted into standards for seismic rehabilitation to facilitate risk reduction. Standards can be selected with varying degrees of risk reduction and varying costs by designating different Rehabilitation Objectives. As described previously, a Rehabilitation Objective is created by specifying a desired Building Performance Level for specified earthquake ground motion criteria. A jurisdiction can thus specify appropriate standards by extracting applicable requirements and incorporating them into its own code or standard, or by reference.

A single Rehabilitation Objective could be selected under all triggering situations (the BSO, for example), or more stringent objectives can be used for important changes to the building, less stringent objectives for minor changes. For example, it is sometimes necessary for design professionals, owners, and building officials to negotiate the extent of seismic improvements done in association with building alterations. Complete rehabilitation is often required by local regulation for complete remodels or major structural alterations. It is the intent of the *Guidelines* to provide a common framework for all of these various uses.

1.6.3 Use in Active or Mandated Programs

Active programs are most often targeted at high-risk building types or occupancies. Active seismic risk reduction programs are those that require owners to rehabilitate their buildings in a certain time frame or, in the case of government agencies or other owners of large inventories, to set self-imposed deadlines for completion.

1.6.3.1 Selection of Buildings to be Included

Programs would logically target only the highest-risk buildings or at least create priorities based on risk. Risk can be based on the likelihood of building failure, the occupancy or importance of buildings, soil types, or other factors. The *Guidelines* are primarily written to be used in the process of rehabilitation and do not directly address the comparative risk level of various building types or other risk factors. Certain building types, such as unreinforced masonry bearing wall buildings and older improperly detailed reinforced concrete frame buildings, have historically presented a high risk, depending on local seismicity and building practice. Therefore, these building types have sometimes been targeted in active programs.

A more pragmatic consideration is the ease of locating targeted buildings. If certain building types cannot be easily identified, either by the local jurisdiction or by the owners and their engineers, enforcement could become difficult and costly. In the extreme, every building designed prior to a given acceptable code cycle would require a seismic evaluation to determine whether targeted characteristics or other risk factors are present, the cost of which may be significant. An alternate procedure might be to select easily identifiable building characteristics to set timelines, even if more accurate building-by-building priorities are somewhat compromised.

1.6.3.2 Selection of Active Seismic Rehabilitation Standards

As discussed for passive programs (Section 1.6.2.2), the *Guidelines* are written to facilitate a wide variation in risk reduction. Factors used to determine an appropriate Rehabilitation Objective include local seismicity, the costs of rehabilitation, and local socioeconomic conditions.

It may be desirable to use Simplified Rehabilitation Methods for active or mandated programs. Only Limited Performance Objectives are included in the *Guidelines* for this method. However, if a program has identified a local building type with few variations in material and configuration, a study of a sample of typical buildings using Systematic Methods may establish that compliance with the requirements of Simplified Rehabilitation meets the BSO, or better, for this building type in this location. Such risk and performance decisions can only be made at the local level.

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2. General Requirements (Simplified and Systematic Rehabilitation)

2.1 Scope

This chapter presents the *Guidelines'* general requirements for rehabilitating existing buildings. The framework in which these requirements are specified is purposefully broad in order to accommodate buildings of many different types, satisfy a broad range of performance levels, and include consideration of the variety of seismic hazards throughout the United States and Territories.

Criteria for the following general issues regarding the seismic rehabilitation of buildings are included in this chapter:

- **Rehabilitation Objectives:** Selection of desired performance levels for given earthquake severity levels
  - **Performance Levels:** Definition of the expected behavior of the building in the design earthquake(s) in terms of limiting levels of damage to the structural and nonstructural components
  - **Seismic Hazard:** Determination of the design ground shaking and other site hazards, such as landsliding, liquefaction, or settlement
- **As-Built Characteristics:** Determination of the basic construction characteristics and earthquake resistive capacity of the existing building
- **Rehabilitation Methods:** Selection of the Simplified or Systematic Method
- **Rehabilitation Strategies:** Selection of a basic strategy for rehabilitation, e.g., providing additional lateral-load-carrying elements, seismic isolation, or reducing the mass of the building
- **Analysis and Design Procedures:** For Systematic Rehabilitation approaches, selection among Linear Static, Linear Dynamic, Nonlinear Static, or Nonlinear Dynamic Procedures
- **General Analysis and Design:** Specification of the force and deformation actions for which given components of a building must be evaluated, and minimum design criteria for interconnection of structural components
- **Building Interaction:** Guidelines for buildings that share elements with neighboring structures, and buildings with performance affected by the presence of adjacent structures
- **Quality Assurance:** Guidelines for ensuring that the design intent is appropriately implemented in the construction process
- **Alternative Materials and Methods:** Guidelines for evaluating and designing structural components not specifically covered by other sections of the *Guidelines*

2.2 Basic Approach

The basic approach for seismic rehabilitation design includes the steps indicated below. Note that these steps are presented here in the order in which they would typically be followed in the rehabilitation process. However, the guidelines for actually performing these steps are presented in a somewhat different order, to facilitate presentation of the concepts.

- Obtain as-built information on the building and determine its characteristics, including whether the building has historic status (Section 2.7).
- Select a Rehabilitation Objective for the building (Section 2.4).
- Select an appropriate Rehabilitation Method (Section 2.8).
- If a Simplified Method is applicable, follow the procedures of Chapter 10; or,
- If a Systematic Method is to be followed:
  - Select a Rehabilitation Strategy (Section 2.10) and perform a preliminary design of corrective measures.
  - Select an appropriate Analysis Procedure (Section 2.9).

– Perform an analysis of the building, including the corrective measures, to verify its adequacy to meet the selected Rehabilitation Objective (Chapter 3).

– If the design is inadequate, revise the corrective measures or try an alternative strategy and repeat the analysis until an acceptable design solution is obtained.

Prior to embarking on a rehabilitation program, an evaluation should be performed to determine whether the building, in its existing condition, has the desired level of seismic resistance. FEMA 178 (BSSC, 1992) is an example of an evaluation methodology that may be used for this purpose. However, FEMA 178 currently does not address objectives other than the Life Safety Performance Level for earthquakes with a 10% probability of exceedance in 50 years (10%/50 year), whereas these *Guidelines* may be used for other performance levels and ground shaking criteria. FEMA 178 is being revised to include the Damage Control Performance Range.

Table 2-1 gives an overview of guidelines and criteria included in this chapter and their relation to guidelines and criteria in other chapters of the *Guidelines*.

2.3 Design Basis

The *Guidelines* are intended to provide a nationally applicable approach for the seismic rehabilitation of buildings. It is expected that most buildings rehabilitated in accordance with the *Guidelines* would perform within the desired levels when subjected to the design earthquakes. However, compliance with the *Guidelines* does not guarantee such performance. The practice of earthquake engineering is rapidly evolving, and both our understanding of the behavior of buildings subjected to strong earthquakes and our ability to predict this behavior are advancing. In the future, new knowledge and technology will provide more reliable methods of accomplishing these goals.

The procedures contained in the *Guidelines* are specifically applicable to the rehabilitation of existing buildings and are, in general, more appropriate for that purpose than are building codes for the seismic design of new buildings. Building codes are primarily intended to regulate the design and construction of new buildings; as such, they include many provisions that encourage the development of designs with features important to good seismic performance, including regular configuration, structural continuity, ductile detailing, and materials of appropriate quality. Many existing buildings were designed and constructed without these features, and contain characteristics—such as unfavorable configuration and poor detailing—that preclude application of building code provisions for their seismic rehabilitation.

A Rehabilitation Objective must be selected as the basis for a rehabilitation design in order to use the provisions of these *Guidelines*. Each Rehabilitation Objective consists of one or more specifications of a seismic demand (hazard level) and corresponding damage state (building performance level). The *Guidelines* present a Basic Safety Objective (BSO), which has performance and hazard levels consistent with seismic risk traditionally considered acceptable in the United States. Alternative objectives that provide lower levels (Limited Objectives) and higher levels (Enhanced Objectives) of performance are also described in the *Guidelines*.

Each structural component and element of the building, including its foundations, shall be classified as either primary or secondary. In a typical building, nearly all elements, including many nonstructural components, will contribute to the building's overall stiffness, mass, and damping, and consequently its response to earthquake ground motion. However, not all of these elements are critical to the ability of the structure to resist collapse when subjected to strong ground shaking. For example, exterior cladding and interior partitions can add substantial initial stiffness to a structure, yet this stiffness is not typically considered in the design of new buildings for lateral force resistance because the lateral strength of these elements is often small. Similarly, the interaction of floor framing systems and columns in shear wall buildings can add some stiffness, although designers typically neglect such stiffness when proportioning the building's shear walls. In the procedures contained in these *Guidelines*, the behavior of all elements and components that participate in the building's lateral response is considered, even if they are not normally considered as part of the lateral-force-resisting system. This is to allow evaluation of the extent of damage likely to be experienced by each of these elements. The concept of primary and secondary elements permits the engineer to differentiate between the performance required of elements that are critical to the building's ability to resist collapse and of those that are not.

Table 2-1 Guidelines and Criteria in Chapter 2 and Relation to Guidelines and Criteria in Other Chapters

	Chapter 2 Criteria Section		Detailed Implementation Criteria in Other Chapters	
Action	**Section**	**Information Presented**	**Chapter(s)**	**Information Presented**
Select Rehabilitation Objective	Section 2.4	Detailed Guidelines		
Select Performance Level	Section 2.5	Detailed Guidelines		
Select Shaking Hazard	Section 2.6	Detailed Criteria		
Evaluate Other Seismic Hazards	Section 2.6	General Discussion	Chapter 4	Evaluation and Mitigation Methods
Obtain As-Built Information, Including Historic Status	Section 2.7	Detailed Criteria	Chapters 4–8 and 11	Material Property Guidelines Testing Guidelines
Select Rehabilitation Method	Section 2.8	Rehabilitation Methods		
Simplified			Chapters 10 and 11	Detailed Guidelines
Systematic	Section 2.11	General Analysis and Design Criteria	Chapters 3–9 and 11	Implementation of Systematic Method
Select Analysis Procedure	Section 2.9	Detailed Criteria		
Select Rehabilitation Strategy	Section 2.10	Detailed Guidelines		
Create Mathematical Model	Section 2.11	General Analysis and Design Criteria	Chapter 3 Chapters 4–9 and 11	Detailed Requirements Stiffness and Strength of Components
Perform Force and Deformation Evaluation	Section 2.11	General Analysis and Design Criteria	Chapter 3	Detailed Criteria
Apply Component Acceptance Criteria	Section 2.9	General Criteria	Chapter 3 Chapters 4–9 and 11	Detailed Criteria Component Strength and Deformation Criteria
Apply Quality Assurance	Section 2.12	Detailed Criteria		
Use Alternative Materials and Methods of Construction	Section 2.13	Detailed Criteria		

- The primary elements and components are those that provide the structure's overall ability to resist collapse under earthquake-induced ground motion. Although damage to these elements, and some degradation of their strength and stiffness, may be permitted to occur, the overall function of these elements in resisting structural collapse should not be compromised.
- Other elements and components of the existing building are designated as secondary. For some structural performance levels, substantial degradation of the lateral-force-resisting stiffness and strength of secondary elements and components is permissible, as this will not inhibit the entire building's capacity to withstand the design ground motions. However, the ability of these secondary elements and components to support gravity loads, under the maximum deformations that the design earthquake(s) would induce in the building, must be preserved.

For a given performance level, acceptance criteria for primary elements and components will typically be more restrictive (i.e., less damage is permissible) than those for secondary elements and components.

In order to comply with the BSO or any Enhanced Rehabilitation Objective, the rehabilitated building shall be provided with a continuous load path, or paths, with adequate strength and stiffness to transfer seismically induced forces caused by ground motion in any direction, from the point of application to the final point of resistance. It shall be demonstrated that all primary and secondary elements of the structure are capable of resisting the forces and deformations corresponding to the earthquake hazards within the acceptance criteria contained in the *Guidelines* for the applicable performance levels. Nonstructural components and building contents shall also be adequately anchored or braced to the structure to control damage as required by the acceptance criteria for the applicable performance level.

2.4 Rehabilitation Objectives

As stated earlier, a Rehabilitation Objective shall be selected as the basis for design. Rehabilitation Objectives are statements of the desired building performance (see Section 2.5) when the building is subjected to earthquake demands of specified severity (see Section 2.6).

Building performance can be described qualitatively in terms of the safety afforded building occupants, during and after the event; the cost and feasibility of restoring the building to pre-earthquake condition; the length of time the building is removed from service to effect repairs; and economic, architectural, or historic impacts on the larger community. These performance characteristics are directly related to the extent of damage sustained by the building.

In these *Guidelines*, the extent of damage to a building is categorized as a Building Performance Level. A broad range of Building Performance Levels may be selected when determining Rehabilitation Objectives. Each Building Performance Level consists of a Structural Performance Level, which defines the permissible damage to structural systems, and a Nonstructural Performance Level, which defines the permissible damage to nonstructural building components and contents.

Section 2.5.1 defines a series of three discrete Structural Performance Levels that may be used in constructing project Rehabilitation Objectives. These are Immediate Occupancy (S-1), Life Safety (S-3), and Collapse Prevention (S-5). Two Structural Performance Ranges are defined to allow design for structural damage states intermediate to those represented by the discrete performance levels. These are Damage Control (S-2) and Limited Safety (S-4). In addition, there is the designation of S-6, Structural Performance Not Considered, to cover the situation where only nonstructural improvements are made.

Section 2.5.2 defines a series of three discrete Nonstructural Performance Levels. These are: Operational Performance Level (N-A), Immediate Occupancy Performance Level (N-B), and Life Safety Performance Level (N-C). There is also a Hazards Reduced Performance Range (N-D) and a fifth level or category (N-E) in which nonstructural damage is not limited.

Section 2.5.3 indicates how Structural and Nonstructural Performance Levels may be combined to form designations for Building Performance Levels. Numerals indicate the Structural Performance Level and letters the Nonstructural Performance Level. Four Performance Levels commonly used in the formation of Building Rehabilitation Objectives are described; these are the Operational Performance Level (1-A), Immediate Performance Occupancy Level (1-B), Life Safety Performance Level (3-C), and Collapse Prevention Performance Level (5-E).

Section 2.6, Seismic Hazard, presents methods for determining earthquake shaking demands and considering other seismic hazards, such as liquefaction and landsliding. Earthquake shaking demands are expressed in terms of ground motion response spectra, discrete parameters that define these spectra, or suites of ground motion time histories, depending on the analysis procedure selected. For sites with significant potential for ground failure, demands should also be expressed in terms of the anticipated permanent differential ground deformations.

Earthquake demands are a function of the location of the building with respect to causative faults, the regional and site-specific geologic characteristics, and the ground motion hazard level(s) selected in the Rehabilitation Objective. In the *Guidelines*, hazard levels may be defined on either a probabilistic or deterministic basis. Probabilistic hazards are defined in terms of the probability that more severe demands will be experienced (probability of exceedance) in a 50-year period. Deterministic demands are defined within a level of confidence in terms of a specific magnitude event on a particular fault, which is most appropriate for buildings located within a few miles of a major active fault. Probabilistic hazard levels frequently used in these *Guidelines* and their corresponding mean return periods (the average number of years between events of similar severity) are as follows:

Earthquake Having Probability of Exceedance	Mean Return Period (years)
50%/50 year	72
20%/50 year	225
10%/50 year	474
2%/50 year	2,475

These mean return periods are typically rounded to 75, 225, 500, and 2,500 years, respectively. The *Guidelines* make frequent reference to two levels of earthquake hazard that are particularly useful for the formation of Rehabilitation Objectives. These are defined in terms of both probabilistic and deterministic approaches. They are termed a Basic Safety Earthquake 1 (BSE-1) and Basic Safety Earthquake 2 (BSE-2). The BSE-1 and BSE-2 earthquakes are typically taken as 10%/50 and 2%/50 year events, respectively, except in regions near major active faults. In these regions the BSE-1 and BSE-2 may be defined based on deterministic estimates of earthquakes on these faults. More detailed discussion of ground motion hazards is presented in Section 2.6.

The Rehabilitation Objective selected as a basis for design will determine, to a great extent, the cost and feasibility of any rehabilitation project, as well as the benefit to be obtained in terms of improved safety, reduction in property damage, and interruption of use in the event of future earthquakes. Table 2-2 presents a matrix indicating the broad range of Rehabilitation Objectives that may be used in these *Guidelines*. (See Section 2.5.3 for definitions of Building Performance Levels.) Each cell in this matrix represents a single Rehabilitation Objective. The goal of a rehabilitation project may be to satisfy a single Rehabilitation Objective—for example, Life Safety for the BSE-1 earthquake—or multiple Rehabilitation Objectives—for example, Life Safety for the BSE-1 earthquake, Collapse Prevention for the BSE-2 earthquake, and Immediate Occupancy for an earthquake with a 50% probability of exceedance in 50 years. A specific

analytical evaluation should be performed to confirm that a rehabilitation design is capable of meeting each desired Rehabilitation Objective selected as a goal for the project.

Table 2-2 Rehabilitation Objectives

		Building Performance Levels			
		Operational Performance Level (1-A)	Immediate Occupancy Performance Level (1-B)	Life Safety Performance Level (3-C)	Collapse Prevention Performance Level (5-E)
Earthquake Hazard Level	50%/50 year	a	b	c	d
	20%/50 year	e	f	g	h
	BSE-1 (~10%/50 year)	i	j	k	l
	BSE-2 (~2%/50 year)	m	n	o	p

k + p = BSO
k + p + any of a, e, i, m; or b, f, j, or n = Enhanced Objectives
o = Enhanced Objective
k alone or p alone = Limited Objectives
c, g, d, h = Limited Objectives

2.4.1 Basic Safety Objective

A desirable goal for rehabilitation is to achieve the Basic Safety Objective (BSO). In order to achieve this objective, building rehabilitation must be designed to achieve both the Life Safety Performance Level (3-C) for BSE-1 earthquake demands and the Collapse Prevention Level (5-E) for BSE-2 earthquake demands. Buildings that have been properly designed and constructed in conformance with the latest edition of the *National Building Code* (BOCA, 1993), *Standard Building Code* (SBCC, 1994), or *Uniform Building*

Code (ICBO, 1994), including all applicable seismic provisions of those codes, may be deemed by code enforcement agencies to meet the BSO.

Building rehabilitation programs designed to the BSO are intended to provide a low risk of danger for any earthquake likely to affect the site. This approximately represents the earthquake risk to life safety traditionally considered acceptable in the United States. Buildings meeting the BSO are expected to experience little damage from the relatively frequent, moderate earthquakes that may occur, but significantly more damage from the most severe and infrequent earthquakes that could affect them.

The level of damage to buildings rehabilitated to the BSO may be greater than that expected in properly designed and constructed new buildings.

When it is desired that a building be able to resist earthquakes with less damage than implied by the BSO, rehabilitation may be designed to one or more of the Enhanced Rehabilitation Objectives of Section 2.4.2.

2.4.2 Enhanced Rehabilitation Objectives

Any Rehabilitation Objective intended to provide performance superior to that of the BSO is termed an Enhanced Objective. An Enhanced Objective must provide better than BSO-designated performance at either the BSE-1 or BSE-2, or both. Enhanced performance can be obtained in two ways:

- Directly, by design for the BSE-1 or BSE-2 earthquakes. Examples include designing for a higher Performance Level than Life Safety for the BSE-1 or a higher Performance Level than Collapse Prevention for the BSE-2.

- Indirectly, by having the design controlled by some other selected Performance Level and hazard that will provide better than BSO performance at the BSE-1 or BSE-2. For example, if providing Immediate Occupancy for a 50%/50 year event controlled the rehabilitation acceptability criteria in such a way that deformation demand were less than that allowed by the BSO, the design would be considered to have an Enhanced Objective.

The *Guidelines* do not incorporate Enhanced Rehabilitation Objectives in any formal procedure, but the definition is included to facilitate discussion of the concept of variable Performance Levels both in the *Guidelines* and the *Commentary*.

2.4.3 Limited Rehabilitation Objectives

Any Rehabilitation Objective intended to provide performance inferior to that of the BSO is termed a Limited Objective. A Limited Objective may consist of either Partial Rehabilitation (Section 2.4.3.1) or Reduced Rehabilitation (Section 2.4.3.2). Limited Rehabilitation Objectives should be permissible if the following conditions are met:

- The rehabilitation measures do not create a structural irregularity or make an existing structural irregularity more severe;

- The rehabilitation measures do not result in a reduction in the capability of the structure to resist lateral forces or deformations;

- The rehabilitation measures do not result in an increase in the seismic forces to any component that does not have adequate capacity to resist these forces, unless this component's behavior is still acceptable considering overall structural performance;

- All new or rehabilitated structural elements are detailed and connected to the existing structure, as required by the *Guidelines*;

- An unsafe condition is not created or made more severe by the rehabilitation measures; and

- Locally adopted and enforced building regulations do not preclude such rehabilitation.

2.4.3.1 Partial Rehabilitation

Any rehabilitation program that does not fully address the lateral-force-resisting capacity of the complete structure is termed Partial Rehabilitation. The portion of the structure that is addressed in Partial Rehabilitation should be designed for a target Rehabilitation Objective and planned so that additional rehabilitation could be performed later to meet fully that objective.

2.4.3.2 Reduced Rehabilitation

Reduced Rehabilitation programs address the entire building's lateral-force-resisting capacity, but not at the levels required for the BSO. Reduced Rehabilitation may be designed for one or more of the following objectives:

- Life Safety Performance Level (3-C) for earthquake demands that are less severe (more probable) than the BSE-1
- Collapse Prevention Performance Level (5-E) for earthquake demands that are less severe (more probable) than the BSE-2
- Performance Levels 4-C, 4-D, 4-E, 5-C, 5-D, 5-E, 6-D, or 6-E for BSE-1 or less severe (more probable) earthquake demands

2.5 Performance Levels

Building performance is a combination of the performance of both structural and nonstructural components. Table 2-3 describes the overall levels of structural and nonstructural damage that may be expected of buildings rehabilitated to the levels defined in the *Guidelines*. For comparative purposes, the estimated performance of a new building subjected to the BSE-1 level of shaking is indicated. These performance descriptions are estimates rather than precise predictions, and variation among buildings of the same Performance Level must be expected.

Independent performance definitions are provided for structural and nonstructural components. Structural performance levels are identified in these *Guidelines* by both a name and numerical designator (following S-) in Section 2.5.1. Nonstructural performance levels are identified by a name and alphabetical designator (following N-) in Section 2.5.2.

2.5.1 Structural Performance Levels and Ranges

Three discrete Structural Performance Levels and two intermediate Structural Performance Ranges are defined. Acceptance criteria, which relate to the permissible earthquake-induced forces and deformations for the various elements of the building, both existing and new, are tied directly to these Structural Performance Ranges and Levels.

A wide range of structural performance requirements could be desired by individual building owners. The three Structural Performance Levels defined in these *Guidelines* have been selected to correlate with the most commonly specified structural performance requirements. The two Structural Performance Ranges permit users with other requirements to customize their building Rehabilitation Objectives.

The Structural Performance Levels are the Immediate Occupancy Level (S-1), the Life Safety Level (S-3), and the Collapse Prevention Level (S-5). Table 2-4 relates these Structural Performance Levels to the limiting damage states for common vertical elements of lateral-force-resisting systems. Table 2-5 relates these Structural Performance Levels to the limiting damage states for common horizontal elements of building lateral-force-resisting systems. Later sections of these *Guidelines* specify design parameters (such as *m* factors, component capacities, and inelastic deformation demands) recommended as limiting values for calculated structural deformations and stresses for different construction components, in order to attain these Structural Performance Levels for a known earthquake demand.

The drift values given in Table 2-4 are typical values provided to illustrate the overall structural response associated with various performance levels. They are not provided in these tables as drift limit requirements of the *Guidelines*, and they do not supersede the specific drift limits or related component or element deformation limits that are specified in Chapters 5 through 9, and 11. The expected post-earthquake state of the buildings described in these tables is for design purposes and should not be used in the post-earthquake safety evaluation process.

The Structural Performance Ranges are the Damage Control Range (S-2) and the Limited Safety Range (S-4). Specific acceptance criteria are not provided for design to these intermediate performance ranges. The engineer wishing to design for such performance needs to determine appropriate acceptance criteria. Acceptance criteria for performance within the Damage Control Range may be obtained by interpolating the acceptance criteria provided for the Immediate Occupancy and Life Safety Performance Levels. Acceptance criteria for performance within the Limited Safety Range may be obtained by interpolating the acceptance criteria for performance within the Life Safety and Collapse Prevention Performance Levels.

2.5.1.1 Immediate Occupancy Performance Level (S-1)

Structural Performance Level S-1, Immediate Occupancy, means the post-earthquake damage state in which only very limited structural damage has occurred. The basic vertical-, and lateral-force-resisting systems of the building retain nearly all of their pre-earthquake strength and stiffness. The risk of life-threatening injury as a result of structural damage is very low, and although some minor structural repairs may be appropriate, these would generally not be required prior to reoccupancy.

2.5.1.2 Life Safety Performance Level (S-3)

Structural Performance Level S-3, Life Safety, means the post-earthquake damage state in which significant damage to the structure has occurred, but some margin against either partial or total structural collapse remains. Some structural elements and components are severely damaged, but this has not resulted in large falling debris hazards, either within or outside the building. Injuries may occur during the earthquake; however, it is expected that the overall risk of life-threatening injury as a result of structural damage is low. It should be possible to repair the structure; however, for economic reasons this may not be practical. While the damaged structure is not an imminent collapse risk, it would be prudent to implement structural repairs or install temporary bracing prior to reoccupancy.

2.5.1.3 Collapse Prevention Performance Level (S-5)

Structural Performance Level S-5, Collapse Prevention, means the building is on the verge of experiencing partial or total collapse. Substantial damage to the structure has occurred, potentially including significant degradation in the stiffness and strength of the lateral-force-resisting system, large permanent lateral deformation of the structure, and—to a more limited extent—degradation in vertical-load-carrying capacity. However, all significant components of the gravity-load-resisting system must continue to carry their gravity load demands. Significant risk of injury due to falling hazards from structural debris may exist. The structure may not be technically practical to repair and is not safe for reoccupancy, as aftershock activity could induce collapse.

2.5.1.4 Damage Control Performance Range (S-2)

Structural Performance Range S-2, Damage Control, means the continuous range of damage states that entail less damage than that defined for the Life Safety level, but more than that defined for the Immediate Occupancy level. Design for Damage Control performance may be desirable to minimize repair time and operation interruption; as a partial means of protecting valuable equipment and contents; or to preserve important historic features when the cost of design for Immediate Occupancy is excessive. Acceptance criteria for this range may be obtained by interpolating between the values provided for the Immediate Occupancy (S-1) and Life Safety (S-3) levels.

2.5.1.5 Limited Safety Performance Range (S-4)

Structural Performance Range S-4, Limited Safety, means the continuous range of damage states between the Life Safety and Collapse Prevention levels. Design parameters for this range may be obtained by interpolating between the values provided for the Life Safety (S-3) and Collapse Prevention (S-5) levels.

2.5.1.6 Structural Performance Not Considered (S-6)

Some owners may desire to address certain nonstructural vulnerabilities in a rehabilitation program—for example, bracing parapets, or anchoring hazardous materials storage containers—without addressing the performance of the structure itself. Such rehabilitation programs are sometimes attractive because they can permit a significant reduction in seismic risk at relatively low cost. The actual performance of the structure with regard to *Guidelines* requirements is not known and could range from a potential collapse hazard to a structure capable of meeting the Immediate Occupancy Performance Level.

2.5.2 Nonstructural Performance Levels

Four Nonstructural Performance Levels are defined in these *Guidelines* and are summarized in Tables 2-6 through 2-8. Nonstructural components addressed in these performance levels include architectural components, such as partitions, exterior cladding, and ceilings; and mechanical and electrical components, including HVAC systems, plumbing, fire suppression systems, and lighting. Occupant contents and furnishings (such as inventory and computers) are included in these tables for some levels but are generally not covered with specific *Guidelines* requirements. Design procedures and acceptance criteria for rehabilitation of nonstructural components to the Life Safety Performance Level are contained in Chapter 11. General guidance only is provided for other performance levels.

2.5.2.1 Operational Performance Level (N-A)

Nonstructural Performance Level A, Operational, means the post-earthquake damage state of the building in which the nonstructural components are able to support the building's intended function. At this level, most nonstructural systems required for normal use of the building—including lighting, plumbing, HVAC, and computer systems—are functional, although minor cleanup and repair of some items may be required. This performance level requires considerations beyond those that are normally within the sole province of the structural engineer. In addition to assuring that nonstructural components are properly mounted and braced within the structure, in order to achieve this performance it is often necessary to provide emergency standby utilities. In addition, it may be necessary to perform rigorous qualification testing of the ability of key electrical and mechanical equipment items to function during or after strong shaking.

Specific design procedures and acceptance criteria for this performance level are not included in the *Guidelines*. Users wishing to design for this performance level will need to refer to appropriate criteria from other sources, such as equipment manufacturers' data, to ensure the performance of mechanical and electrical systems.

2.5.2.2 Immediate Occupancy Level (N-B)

Nonstructural Performance Level B, Immediate Occupancy, means the post-earthquake damage state in which only limited nonstructural damage has occurred. Basic access and life safety systems, including doors, stairways, elevators, emergency lighting, fire alarms, and suppression systems, remain operable, provided that power is available. There could be minor window breakage and slight damage to some components. Presuming that the building is structurally safe, it is expected that occupants could safely remain in the building, although normal use may be impaired and some cleanup and inspection may be required. In general, components of mechanical and electrical systems in the building are structurally secured and should be able to function if necessary utility service is available. However, some components may experience misalignments or internal damage and be nonoperable. Power, water, natural gas, communications lines, and other utilities required for normal building use may not be available. The risk of life-threatening injury due to nonstructural damage is very low.

2.5.2.3 Life Safety Level (N-C)

Nonstructural Performance Level C, Life Safety, is the post-earthquake damage state in which potentially significant and costly damage has occurred to nonstructural components but they have not become dislodged and fallen, threatening life safety either within or outside the building. Egress routes within the building are not extensively blocked, but may be impaired by lightweight debris. HVAC, plumbing, and fire suppression systems may have been damaged, resulting in local flooding as well as loss of function. While injuries may occur during the earthquake from the failure of nonstructural components, it is expected that, overall, the risk of life-threatening injury is very low. Restoration of the nonstructural components may take extensive effort.

2.5.2.4 Hazards Reduced Level (N-D)

Nonstructural Performance Level D, Hazards Reduced, represents a post-earthquake damage state level in which extensive damage has occurred to nonstructural components, but large or heavy items that pose a falling hazard to a number of people—such as parapets, cladding panels, heavy plaster ceilings, or storage racks— are prevented from falling. While isolated serious injury could occur from falling debris, failures that could injure large numbers of persons—either inside or outside the structure—should be avoided. Exits, fire suppression systems, and similar life-safety issues are not addressed in this performance level.

2.5.2.5 Nonstructural Performance Not Considered (N-E)

In some cases, the decision may be made to rehabilitate the structure without addressing the vulnerabilities of nonstructural components. It may be desirable to do this when rehabilitation must be performed without interruption of building operation. In some cases, it is possible to perform all or most of the structural rehabilitation from outside occupied building areas, while extensive disruption of normal operation may be required to perform nonstructural rehabilitation. Also, since many of the most severe hazards to life safety occur as a result of structural vulnerabilities, some municipalities may wish to adopt rehabilitation ordinances that require structural rehabilitation only.

2.5.3 Building Performance Levels

Building Performance Levels are obtained by combining Structural and Nonstructural Performance Levels. A large number of combinations is possible. Each Building Performance Level is designated alpha-numerically with a numeral representing the Structural Performance Level and a letter representing the Nonstructural Performance Level (e.g. 1-B, 3-C). Table 2-9 indicates the possible combinations and provides names for those that are most likely to be selected as a basis for design. Several of the more common Building Performance Levels are described below.

2.5.3.1 Operational Level (1-A)

This Building Performance Level is a combination of the Structural Immediate Occupancy Level and the Nonstructural Operational Level. Buildings meeting this performance level are expected to sustain minimal or no damage to their structural and nonstructural components. The building is suitable for its normal occupancy and use, although possibly in a slightly impaired mode, with power, water, and other required utilities provided from emergency sources, and possibly with some nonessential systems not functioning. Buildings meeting this performance level pose an extremely low risk to life safety.

Under very low levels of earthquake ground motion, most buildings should be able to meet or exceed this performance level. Typically, however, it will not be economically practical to design for this performance under severe levels of ground shaking, except for buildings that house essential services.

2.5.3.2 Immediate Occupancy Level (1-B)

This Building Performance Level is a combination of the Structural and Nonstructural Immediate Occupancy levels. Buildings meeting this performance level are expected to sustain minimal or no damage to their structural elements and only minor damage to their nonstructural components. While it would be safe to reoccupy a building meeting this performance level immediately following a major earthquake, nonstructural systems may not function due to either a lack of electrical power or internal damage to equipment. Therefore, although immediate reoccupancy of the building is possible, it may be necessary to perform some cleanup and repair, and await the restoration of utility service, before the building could function in a normal mode. The risk to life safety at this performance level is very low.

Many building owners may wish to achieve this level of performance when the building is subjected to moderate levels of earthquake ground motion. In addition, some owners may desire such performance for very important buildings, under severe levels of earthquake ground shaking. This level provides most of the protection obtained under the Operational Level, without the cost of providing standby utilities and performing rigorous seismic qualification of equipment performance.

2.5.3.3 Life Safety Level (3-C)

This Building Performance Level is a combination of the Structural and Nonstructural Life Safety levels. Buildings meeting this level may experience extensive damage to structural and nonstructural components. Repairs may be required before reoccupancy of the building occurs, and repair may be deemed economically impractical. The risk to life in buildings meeting this performance level is low.

This performance level entails somewhat more damage than anticipated for new buildings that have been properly designed and constructed for seismic resistance when subjected to their design earthquakes. Many building owners will desire to meet this performance level for a severe level of ground shaking.

2.5.3.4 Collapse Prevention Level (5-E)

This Building Performance Level consists of the Structural Collapse Prevention Level with no consideration of nonstructural vulnerabilities, except that parapets and heavy appendages are rehabilitated. Buildings meeting this performance level may pose a significant hazard to life safety resulting from failure of nonstructural components. However, because the building itself does not collapse, gross loss of life should be avoided. Many buildings meeting this level will be complete economic losses.

This level has sometimes been selected as the basis for mandatory seismic rehabilitation ordinances enacted by municipalities, as it results in mitigation of the most severe life-safety hazards at relatively low cost.

Table 2-3 Damage Control and Building Performance Levels

	Building Performance Levels			
	Collapse Prevention Level	**Life Safety Level**	**Immediate Occupancy Level**	**Operational Level**
Overall Damage	Severe	Moderate	Light	Very Light
General	Little residual stiffness and strength, but load-bearing columns and walls function. Large permanent drifts. Some exits blocked. Infills and unbraced parapets failed or at incipient failure. Building is near collapse.	Some residual strength and stiffness left in all stories. Gravity-load-bearing elements function. No out-of-plane failure of walls or tipping of parapets. Some permanent drift. Damage to partitions. Building may be beyond economical repair.	No permanent drift. Structure substantially retains original strength and stiffness. Minor cracking of facades, partitions, and ceilings as well as structural elements. Elevators can be restarted. Fire protection operable.	No permanent drift; structure substantially retains original strength and stiffness. Minor cracking of facades, partitions, and ceilings as well as structural elements. All systems important to normal operation are functional.
Nonstructural components	Extensive damage.	Falling hazards mitigated but many architectural, mechanical, and electrical systems are damaged.	Equipment and contents are generally secure, but may not operate due to mechanical failure or lack of utilities.	Negligible damage occurs. Power and other utilities are available, possibly from standby sources.
Comparison with performance intended for buildings designed, under the *NEHRP Provisions,* for the Design Earthquake	Significantly more damage and greater risk.	Somewhat more damage and slightly higher risk.	Much less damage and lower risk.	Much less damage and lower risk.

Table 2-4 Structural Performance Levels and Damage[1]—Vertical Elements

		Structural Performance Levels		
Elements	**Type**	**Collapse Prevention S-5**	**Life Safety S-3**	**Immediate Occupancy S-1**
Concrete Frames	Primary	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some nonductile columns. Severe damage in short columns.	Extensive damage to beams. Spalling of cover and shear cracking (< 1/8" width) for ductile columns. Minor spalling in nonductile columns. Joint cracks < 1/8" wide.	Minor hairline cracking. Limited yielding possible at a few locations. No crushing (strains below 0.003).
	Secondary	Extensive spalling in columns (limited shortening) and beams. Severe joint damage. Some reinforcing buckled.	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some nonductile columns. Severe damage in short columns.	Minor spalling in a few places in ductile columns and beams. Flexural cracking in beams and columns. Shear cracking in joints < 1/16" width.
	Drift[2]	4% transient or permanent	2% transient; 1% permanent	1% transient; negligible permanent

Table 2-4 Structural Performance Levels and Damage[1]—Vertical Elements (continued)

Elements	Type	Structural Performance Levels		
		Collapse Prevention S-5	**Life Safety S-3**	**Immediate Occupancy S-1**
Steel Moment Frames	Primary	Extensive distortion of beams and column panels. Many fractures at moment connections, but shear connections remain intact.	Hinges form. Local buckling of some beam elements. Severe joint distortion; isolated moment connection fractures, but shear connections remain intact. A few elements may experience partial fracture.	Minor local yielding at a few places. No fractures. Minor buckling or observable permanent distortion of members.
	Secondary	Same as primary.	Extensive distortion of beams and column panels. Many fractures at moment connections, but shear connections remain intact.	Same as primary.
	Drift[2]	5% transient or permanent	2.5% transient; 1% permanent	0.7% transient; negligible permanent
Braced Steel Frames	Primary	Extensive yielding and buckling of braces. Many braces and their connections may fail.	Many braces yield or buckle but do not totally fail. Many connections may fail.	Minor yielding or buckling of braces.
	Secondary	Same as primary.	Same as primary.	Same as primary.
	Drift[2]	2% transient or permanent	1.5% transient; 0.5% permanent	0.5% transient; negligible permanent
Concrete Walls	Primary	Major flexural and shear cracks and voids. Sliding at joints. Extensive crushing and buckling of reinforcement. Failure around openings. Severe boundary element damage. Coupling beams shattered and virtually disintegrated.	Some boundary element distress, including limited buckling of reinforcement. Some sliding at joints. Damage around openings. Some crushing and flexural cracking. Coupling beams: extensive shear and flexural cracks; some crushing, but concrete generally remains in place.	Minor hairline cracking of walls, < 1/16" wide. Coupling beams experience cracking < 1/8" width.
	Secondary	Panels shattered and virtually disintegrated.	Major flexural and shear cracks. Sliding at joints. Extensive crushing. Failure around openings. Severe boundary element damage. Coupling beams shattered and virtually disintegrated.	Minor hairline cracking of walls. Some evidence of sliding at construction joints. Coupling beams experience cracks < 1/8" width. Minor spalling.
	Drift[2]	2% transient or permanent	1% transient; 0.5% permanent	0.5% transient; negligible permanent

Table 2-4 Structural Performance Levels and Damage[1]—Vertical Elements (continued)

Elements	Type	Structural Performance Levels		
		Collapse Prevention S-5	**Life Safety S-3**	**Immediate Occupancy S-1**
Unreinforced Masonry Infill Walls[3]	Primary	Extensive cracking and crushing; portions of face course shed.	Extensive cracking and some crushing but wall remains in place. No falling units. Extensive crushing and spalling of veneers at corners of openings.	Minor (<1/8" width) cracking of masonry infills and veneers. Minor spalling in veneers at a few corner openings.
	Secondary	Extensive crushing and shattering; some walls dislodge.	Same as primary.	Same as primary.
	Drift[2]	0.6% transient or permanent	0.5% transient; 0.3% permanent	0.1% transient; negligible permanent
Unreinforced Masonry (Noninfill) Walls	Primary	Extensive cracking; face course and veneer may peel off. Noticeable in-plane and out-of-plane offsets.	Extensive cracking. Noticeable in-plane offsets of masonry and minor out-of-plane offsets.	Minor (< 1/8" width) cracking of veneers. Minor spalling in veneers at a few corner openings. No observable out-of-plane offsets.
	Secondary	Nonbearing panels dislodge.	Same as primary.	Same as primary.
	Drift[2]	1% transient or permanent	0.6% transient; 0.6% permanent	0.3% transient; 0.3% permanent
Reinforced Masonry Walls	Primary	Crushing; extensive cracking. Damage around openings and at corners. Some fallen units.	Extensive cracking (< 1/4") distributed throughout wall. Some isolated crushing.	Minor (< 1/8" width) cracking. No out-of-plane offsets.
	Secondary	Panels shattered and virtually disintegrated.	Crushing; extensive cracking; damage around openings and at corners; some fallen units.	Same as primary.
	Drift[2]	1.5% transient or permanent	0.6% transient; 0.6% permanent	0.2% transient; 0.2% permanent
Wood Stud Walls	Primary	Connections loose. Nails partially withdrawn. Some splitting of members and panels. Veneers dislodged.	Moderate loosening of connections and minor splitting of members.	Distributed minor hairline cracking of gypsum and plaster veneers.
	Secondary	Sheathing sheared off. Let-in braces fractured and buckled. Framing split and fractured.	Connections loose. Nails partially withdrawn. Some splitting of members and panels.	Same as primary.
	Drift[2]	3% transient or permanent	2% transient; 1% permanent	1% transient; 0.25% permanent

Table 2-4 Structural Performance Levels and Damage[1]—Vertical Elements (continued)

Elements	Type	Structural Performance Levels Collapse Prevention S-5	 Life Safety S-3	 Immediate Occupancy S-1
Precast Concrete Connections	Primary	Some connection failures but no elements dislodged.	Local crushing and spalling at connections, but no gross failure of connections.	Minor working at connections; cracks < 1/16" width at connections.
	Secondary	Same as primary.	Some connection failures but no elements dislodged.	Minor crushing and spalling at connections.
Foundations	General	Major settlement and tilting.	Total settlements < 6" and differential settlements < 1/2" in 30 ft.	Minor settlement and negligible tilting.

1. The damage states indicated in this table are provided to allow an understanding of the severity of damage that may be sustained by various structural elements when present in structures meeting the definitions of the Structural Performance Levels. These damage states are not intended for use in post-earthquake evaluation of damage nor for judging the safety of, or required level of repair to, a structure following an earthquake.
2. The drift values, differential settlements, and similar quantities indicated in these tables are not intended to be used as acceptance criteria for evaluating the acceptability of a rehabilitation design in accordance with the analysis procedures provided in these *Guidelines*; rather, they are indicative of the range of drift that typical structures containing the indicated structural elements may undergo when responding within the various performance levels. Drift control of a rehabilitated structure may often be governed by the requirements to protect nonstructural components. Acceptable levels of foundation settlement or movement are highly dependent on the construction of the superstructure. The values indicated are intended to be qualitative descriptions of the approximate behavior of structures meeting the indicated levels.
3. For limiting damage to frame elements of infilled frames, refer to the rows for concrete or steel frames.

Table 2-5 Structural Performance Levels and Damage—Horizontal Elements

Element	Performance Levels Collapse Prevention S-5	 Life Safety S-3	 Immediate Occupancy S-1
Metal Deck Diaphragms	Large distortion with buckling of some units and tearing of many welds and seam attachments.	Some localized failure of welded connections of deck to framing and between panels. Minor local buckling of deck.	Connections between deck units and framing intact. Minor distortions.
Wood Diaphragms	Large permanent distortion with partial withdrawal of nails and extensive splitting of elements.	Some splitting at connections. Loosening of sheathing. Observable withdrawal of fasteners. Splitting of framing and sheathing.	No observable loosening or withdrawal of fasteners. No splitting of sheathing or framing.
Concrete Diaphragms	Extensive crushing and observable offset across many cracks.	Extensive cracking (< 1/4" width). Local crushing and spalling.	Distributed hairline cracking. Some minor cracks of larger size (< 1/8" width).
Precast Diaphragms	Connections between units fail. Units shift relative to each other. Crushing and spalling at joints.	Extensive cracking (< 1/4" width). Local crushing and spalling.	Some minor cracking along joints.

Table 2-6 Nonstructural Performance Levels and Damage—Architectural Components

Component	Nonstructural Performance Levels			
	Hazards Reduced Level N-D	**Life Safety N-C**	**Immediate Occupancy N-B**	**Operational N-A**
Cladding	Severe damage to connections and cladding. Many panels loosened.	Severe distortion in connections. Distributed cracking, bending, crushing, and spalling of cladding elements. Some fracturing of cladding, but panels do not fall.	Connections yield; minor cracks (< 1/16" width) or bending in cladding.	Connections yield; minor cracks (< 1/16" width) or bending in cladding.
Glazing	General shattered glass and distorted frames. Widespread falling hazards.	Extensive cracked glass; little broken glass.	Some cracked panes; none broken.	Some cracked panes; none broken
Partitions	Severe racking and damage in many cases.	Distributed damage; some severe cracking, crushing, and racking in some areas.	Cracking to about 1/16" width at openings. Minor crushing and cracking at corners.	Cracking to about 1/16" width at openings. Minor crushing and cracking at corners.
Ceilings	Most ceilings damaged. Light suspended ceilings dropped. Severe cracking in hard ceilings.	Extensive damage. Dropped suspended ceiling tiles. Moderate cracking in hard ceilings.	Minor damage. Some suspended ceiling tiles disrupted. A few panels dropped. Minor cracking in hard ceilings.	Generally negligible damage. Isolated suspended panel dislocations, or cracks in hard ceilings.
Parapets and Ornamentation	Extensive damage; some fall in nonoccupied areas.	Extensive damage; some falling in nonoccupied areas.	Minor damage.	Minor damage.
Canopies & Marquees	Extensive distortion.	Moderate distortion.	Minor damage.	Minor damage.
Chimneys & Stacks	Extensive damage. No collapse.	Extensive damage. No collapse.	Minor cracking.	Negligible damage.
Stairs & Fire Escapes	Extensive racking. Loss of use.	Some racking and cracking of slabs, usable.	Minor damage.	Negligible damage.
Light Fixtures	Extensive damage. Falling hazards occur.	Many broken light fixtures. Falling hazards generally avoided in heavier fixtures (> 20 pounds).	Minor damage. Some pendant lights broken.	Negligible damage.
Doors	Distributed damage. Many racked and jammed doors.	Distributed damage. Some racked and jammed doors.	Minor damage. Doors operable.	Minor damage. Doors operable.

Table 2-7 Nonstructural Performance Levels and Damage—Mechanical, Electrical, and Plumbing Systems/Components

	Nonstructural Performance Levels			
System/Component	**Hazards Reduced N-D**	**Life Safety N-C**	**Immediate Occupancy N-B**	**Operational N-A**
Elevators	Elevators out of service; counterweights off rails.	Elevators out of service; counterweights do not dislodge.	Elevators operable; can be started when power available.	Elevators operate.
HVAC Equipment	Most units do not operate; many slide or overturn; some suspended units fall.	Units shift on supports, rupturing attached ducting, piping, and conduit, but do not fall.	Units are secure and most operate if power and other required utilities are available.	Units are secure and operate; emergency power and other utilities provided, if required.
Ducts	Ducts break loose of equipment and louvers; some supports fail; some ducts fall.	Minor damage at joints of sections and attachment to equipment; some supports damaged, but ducts do not fall.	Minor damage at joints, but ducts remain serviceable.	Negligible damage.
Piping	Some lines rupture. Some supports fail. Some piping falls.	Minor damage at joints, with some leakage. Some supports damaged, but systems remain suspended.	Minor leaks develop at a few joints.	Negligible damage.
Fire Sprinkler Systems	Many sprinkler heads damaged by collapsing ceilings. Leaks develop at couplings. Some branch lines fail.	Some sprinkler heads damaged by swaying ceilings. Leaks develop at some couplings.	Minor leakage at a few heads or pipe joints. System remains operable.	Negligible damage.
Fire Alarm Systems	Ceiling mounted sensors damaged. System nonfunctional.	May not function.	System is functional.	System is functional.
Emergency Lighting	Some lights fall. Power may not be available.	System is functional.	System is functional.	System is functional.
Electrical Distribution Equipment	Units slide and/or overturn, rupturing attached conduit. Uninterruptable Power Source systems fail. Diesel generators do not start.	Units shift on supports and may not operate. Generators provided for emergency power start; utility service lost.	Units are secure and generally operable. Emergency generators start, but may not be adequate to service all power requirements.	Units are functional. Emergency power is provided, as needed.
Plumbing	Some fixtures broken; lines broken; mains disrupted at source.	Some fixtures broken, lines broken; mains disrupted at source.	Fixtures and lines serviceable; however, utility service may not be available.	System is functional. On-site water supply provided, if required.

Table 2-8 Nonstructural Performance Levels and Damage—Contents

Contents Type	Nonstructural Performance Levels			
	Hazards Reduced N-D	**Life Safety N-C**	**Immediate Occupancy N-B**	**Operational N-A**
Computer Systems	Units roll and overturn, disconnect cables. Raised access floors collapse.	Units shift and may disconnect cables, but do not overturn. Power not available.	Units secure and remain connected. Power may not be available to operate, and minor internal damage may occur.	Units undamaged and operable; power available.
Manufacturing Equipment	Units slide and overturn; utilities disconnected. Heavy units require reconnection and realignment. Sensitive equipment may not be functional.	Units slide, but do not overturn; utilities not available; some realignment required to operate.	Units secure, and most operable if power and utilities available.	Units secure and operable; power and utilities available.
Desktop Equipment	Units slide off desks.	Some equipment slides off desks.	Some equipment slides off desks.	Equipment secured to desks and operable.
File Cabinets	Cabinets overturn and spill contents.	Drawers slide open; cabinets tip.	Drawers slide open, but cabinets do not tip.	Drawers slide open, but cabinets do not tip.
Book Shelves	Shelves overturn and spill contents.	Books slide off shelves.	Books slide on shelves.	Books remain on shelves.
Hazardous Materials	Severe damage; no large quantity of material released.	Minor damage; occasional materials spilled; gaseous materials contained.	Negligible damage; materials contained.	Negligible damage; materials contained.
Art Objects	Objects damaged by falling, water, dust.	Objects damaged by falling, water, dust.	Some objects may be damaged by falling.	Objects undamaged.

Table 2-9 Building Performance Levels/Ranges

Nonstructural Performance Levels	Structural Performance Levels/Ranges					
	S-1 Immediate Occupancy	S-2 Damage Control Range	S-3 Life Safety	S-4 Limited Safety Range	S-5 Collapse Prevention	S-6 Not Considered
N-A Operational	Operational 1-A	2-A	Not recommended	Not recommended	Not recommended	Not recommended
N-B Immediate Occupancy	Immediate Occupancy 1-B	2-B	3-B	Not recommended	Not recommended	Not recommended
N-C Life Safety	1-C	2-C	Life Safety 3-C	4-C	5-C	6-C
N-D Hazards Reduced	Not recommended	2-D	3-D	4-D	5-D	6-D
N-E Not Considered	Not recommended	Not recommended	Not recommended	4-E	5-E Collapse Prevention	No rehabilitation

2.6 Seismic Hazard

The most common and significant cause of earthquake damage to buildings is ground shaking; thus, the effects of ground shaking form the basis for most building code requirements for seismic design. As stated in Section 2.4, two levels of earthquake shaking hazard are used to satisfy the BSO for these *Guidelines*. These are termed Basic Safety Earthquake 1 (BSE-1) and Basic Safety Earthquake 2 (BSE-2). BSE-2 earthquake ground shaking, also termed Maximum Considered Earthquake (MCE) ground shaking, is similar to that defined for the MCE in the 1997 *NEHRP Recommended Provisions* (BSSC, 1997). In most areas of the United States, BSE-2 earthquake ground motion has a 2% probability of exceedance in 50 years (2%/50 year). In regions close to known faults with significant slip rates and characteristic earthquakes with magnitudes in excess of about 6.0, the BSE-2 ground shaking is limited by a conservative estimate (150% of the median attenuation) of the shaking likely to be experienced as a result of such a characteristic event. Ground shaking levels determined in this manner will typically correspond to a probability of exceedance that is greater than 2% in 50 years. The BSE-1 earthquake is similar, but not identical to the concept of a design earthquake contained in the *NEHRP Provisions*. It is defined as that ground shaking having a 10% probability of exceedance in 50 years (10%/50 year). The motions need not exceed those used for new buildings, defined as 2/3 of the BSE-2 motion.

In addition to the BSE-1 and BSE-2 levels of ground motion, Rehabilitation Objectives may be formed considering earthquake ground shaking hazards with any defined probability of exceedance, or based on any deterministic event on a specific fault.

Response spectra are used to characterize earthquake shaking demand on buildings in the *Guidelines*. Ground shaking response spectra for use in seismic rehabilitation design may be determined in accordance with either the General Procedure of Section 2.6.1 or the Site-Specific Procedure of Section 2.6.2. Seismic zones are defined in Section 2.6.3. Other seismic hazards (e.g., liquefaction) are discussed in Section 2.6.4.

In the General Procedure, ground shaking hazard is determined from available response spectrum acceleration contour maps. Maps showing 5%-damped response spectrum ordinates for short-period (0.2 second) and long-period (1 second) response distributed with the *Guidelines* can be used directly with the General Procedure of Section 2.6.1 for developing design response spectra for either or both the BSE-1 and BSE-2, or for earthquakes of any desired probability of exceedance. Alternatively, other maps and other procedures can be used, provided that 5%-damped response spectra are developed that represent the ground shaking for the desired earthquake return period, and the site soil classification is considered. In the Site-Specific Procedure, ground shaking hazard is determined using a specific study of the faults and seismic source zones that may affect the site, as well as evaluation of the regional and geologic conditions that affect the character of the site ground motion caused by events occurring on these faults and sources.

The General Procedure may be used for any building. The Site-Specific Procedure may also be used for any building and should be considered where any of the following apply:

- Rehabilitation is planned to an Enhanced Rehabilitation Objective, as defined in Section 2.4.2.
- The building site is located within 10 kilometers of an active fault.
- The building is located on Type E soils (as defined in Section 2.6.1.4) and the mapped BSE-2 spectral response acceleration at short periods (S_S) exceeds 2.0g.
- The building is located on Type F soils as defined in Section 2.6.1.4.

  Exception: Where S_S, determined in accordance with Section 2.6.1.1, < 0.20g. In these cases, a Type E soil profile may be assumed.
- A time history response analysis of the building will be performed as part of the design.

Other site-specific seismic hazards that may cause damage to buildings include:

- surface fault rupture
- differential compaction of the foundation material
- landsliding

- liquefaction
- lateral spreading
- flooding

If the potential for any of these, or other, seismic hazards exists at a given site, then they also should be considered in the rehabilitation design, in accordance with Section 2.6.4 and Chapter 4.

2.6.1 General Ground Shaking Hazard Procedure

The general procedures of this section may be used to determine acceleration response spectra for any of the following hazard levels:

- Basic Safety Earthquake 1 (BSE-1)
- Basic Safety Earthquake 2 (BSE-2)
- Earthquake with any defined probability of exceedance in 50 years

Deterministic estimates of earthquake hazard, in which an acceleration response spectrum is obtained for a specific magnitude earthquake occurring on a defined fault, shall be made using the Site-Specific Procedures of Section 2.6.2.

The basic steps for determining a response spectrum under this general procedure are:s

1. Determine whether the desired hazard level corresponds to one of the levels contained in the ground shaking hazard maps distributed with the *Guidelines*. The package includes maps for BSE-2 (MCE) ground shaking hazards as well as for hazards with 10%/50 year exceedance probabilities.
2. If the desired hazard level corresponds with one of the mapped hazard levels, obtain spectral response acceleration parameters directly from the maps, in accordance with Section 2.6.1.1.
3. If the desired hazard level is the BSE-1, then obtain the spectral response acceleration parameters from the maps, in accordance with Section 2.6.1.2.
4. If the desired hazard level does not correspond with the mapped levels of hazard, then obtain the spectral response acceleration parameters from the available maps, and modify them to the desired hazard level, either by logarithmic interpolation or extrapolation, in accordance with Section 2.6.1.3.
5. Obtain design spectral response acceleration parameters by adjusting the mapped, or modified mapped spectral response acceleration parameters for site class effects, in accordance with Section 2.6.1.4.
6. Using the design spectral response acceleration parameters that have been adjusted for site class effects, construct the response spectrum in accordance with Section 2.6.1.5.

2.6.1.1 BSE-2 and 10%/50 Response Acceleration Parameters

The mapped short-period response acceleration parameter, S_S, and mapped response acceleration parameter at a one-second period, S_1, for BSE-2 ground motion hazards may be obtained directly from the maps distributed with the *Guidelines*. The mapped short-period response acceleration parameter, S_S, and mapped response acceleration parameter at a one-second period, S_1, for 10%/50 year ground motion hazards may also be obtained directly from the maps distributed with the *Guidelines*.

Parameters S_S and S_1 shall be obtained by interpolating between the values shown on the response acceleration contour lines on either side of the site, on the appropriate map, or by using the value shown on the map for the higher contour adjacent to the site.

2.6.1.2 BSE-1 Response Acceleration Parameters

The mapped short-period response acceleration parameter, S_S, and mapped response acceleration parameter at a one-second period, S_1, for BSE-1 ground shaking hazards shall be taken as the smaller of the following:

- The values of the parameters S_S and S_1, respectively, determined for 10%/50 year ground motion hazards, in accordance with Section 2.6.1.1.
- Two thirds of the values of the parameters S_S and S_1, respectively, determined for BSE-2 ground motion hazards, in accordance with Section 2.6.1.1.

2.6.1.3 Adjustment of Mapped Response Acceleration Parameters for Other Probabilities of Exceedance

When the mapped BSE-2 short period response acceleration parameter, S_S, is less than 1.5g, the modified mapped short period response acceleration parameter, S_S, and modified mapped response acceleration parameter at a one-second period, S_1, for probabilities of exceedance between 2%/50 years and 10%/50 years may be determined from the equation:

$$1n(S_i) = 1n(S_{i10/50}) + [1n(S_{iBSE-2}) - 1n(S_{i10/50})] [0.606\ 1n(P_R) - 3.73] \quad (2\text{-}1)$$

where:

$1n(S_i)$	=	Natural logarithm of the spectral acceleration parameter ("i" = "s" for short period or "i" = 1 for 1 second period) at the desired probability of exceedance
$1n(S_{i10/50})$	=	Natural logarithm of the spectral acceleration parameter ("i" = "s" for short period or "i" = 1 for 1 second period) at a 10%/50 year exceedance rate
$1n(S_{iBSE-2})$	=	Natural logarithm of the spectral acceleration parameter ("i" = "s" for short period or "i" = 1 for 1 second period) for the BSE-2 hazard level
$1n(P_R)$	=	Natural logarithm of the mean return period corresponding to the exceedance probability of the desired hazard level

and the mean return period P_R at the desired exceedance probability may be calculated from the equation:

$$P_R = \frac{1}{1 - e^{0.02\ 1n(1 - P_{E50})}} \quad (2\text{-}2)$$

where P_{E50} is the probability of exceedance in 50 years of the desired hazard level.

When the mapped BSE-2 short period response acceleration parameter, S_S, is greater than or equal to 1.5g, the modified mapped short period response acceleration parameter, S_S, and modified mapped response acceleration parameter at a one-second period, S_1, for probabilities of exceedance between 2%/50 years and 10%/50 years may be determined from the equation:

$$S_i = S_{i10/50}\left(\frac{P_R}{475}\right)^n \quad (2\text{-}3)$$

where S_i, $S_{i10/50}$, and P_R are as defined above and n may be obtained from Table 2-10.

Table 2-10 and the two following specify five regions, three of which are not yet specifically defined, namely Intermountain, Central US, and Eastern US. For states or areas that might lie near the regional borders, care will be necessary.

Table 2-10 Values of Exponent n for Determination of Response Acceleration Parameters at Hazard Levels between 10%/50 years and 2%/50 years; Sites where Mapped BSE-2 Values of $S_S \geq 1.5g$

	Values of Exponent *n* for	
Region	S_S	S_1
California	0.29	0.29
Pacific Northwest	0.56	0.67
Intermountain	0.50	0.60
Central US	0.98	1.09
Eastern US	0.93	1.05

When the mapped BSE-2 short period response acceleration parameter, S_S, is less than 1.5g, the modified mapped short period response acceleration parameter, S_S, and modified mapped response acceleration parameter at a one-second period, S_1, for probabilities of exceedance greater than 10%/50 years may be determined from Equation 2-3, where the exponent n is obtained from Table 2-11.

When the mapped BSE-2 short period response acceleration parameter, S_S, is greater than or equal to 1.5g, the modified mapped short period response acceleration parameter, S_S, and modified mapped response acceleration parameter at a one-second period, S_1, for probabilities of exceedance greater than 10%/50

Table 2-11 Values of Exponent n for Determination of Response Acceleration Parameters at Probabilities of Exceedance Greater than 10%/50 years; Sites where Mapped BSE-2 Values of S_S < 1.5g

	Values of Exponent *n* for	
Region	S_S	S_1
California	0.44	0.44
Pacific Northwest and Intermountain	0.54	0.59
Central and Eastern US	0.77	0.80

Table 2-12 Values of Exponent n for Determination of Response Acceleration Parameters at Probabilities of Exceedance Greater than 10%/50 years; Sites where Mapped BSE-2 Values of $S_S \geq$ 1.5g

	Values of Exponent *n* for	
Region	S_S	S_1
California	0.44	0.44
Pacific Northwest	0.89	0.96
Intermountain	0.54	0.59
Central US	0.89	0.89
Eastern US	1.25	1.25

years may be determined from Equation 2-3, where the exponent *n* is obtained from Table 2-12.

2.6.1.4 Adjustment for Site Class

The design short-period spectral response acceleration parameter, S_{XS}, and the design spectral response acceleration parameter at one second, S_{X1}, shall be obtained respectively from Equations 2-4 and 2-5 as follows:

$$S_{XS} = F_a S_S \tag{2-4}$$

$$S_{X1} = F_v S_1 \tag{2-5}$$

where F_a and F_v are site coefficients determined respectively from Tables 2-13 and 2-14, based on the site class and the values of the response acceleration parameters S_S and S_1.

Table 2-13 Values of F_a as a Function of Site Class and Mapped Short-Period Spectral Response Acceleration S_S

	Mapped Spectral Acceleration at Short Periods S_S				
Site Class	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	*
F	*	*	*	*	*

NOTE: Use straight-line interpolation for intermediate values of S_S.

* Site-specific geotechnical investigation and dynamic site response analyses should be performed.

Table 2-14 Values of F_v as a Function of Site Class and Mapped Spectral Response Acceleration at One-Second Period S_1

	Mapped Spectral Acceleration at One-Second Period S_1				
Site Class	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	*
F	*	*	*	*	*

NOTE: Use straight-line interpolation for intermediate values of S_1.

* Site-specific geotechnical investigation and dynamic site response analyses should be performed.

Site classes shall be defined as follows:

- **Class A:** Hard rock with measured shear wave velocity, $\bar{v}_s$ > 5,000 ft/sec
- **Class B:** Rock with 2,500 ft/sec < $\bar{v}_s$ < 5,000 ft/sec

- **Class C:** Very dense soil and soft rock with 1,200 ft/sec < $\bar{v}_s \le$ 2,500 ft/sec or with either standard blow count $\bar{N}$ > 50 or undrained shear strength $\bar{s}_u$ > 2,000 psf
- **Class D:** Stiff soil with 600 ft/sec < $\bar{v}_s \le$ 1,200 ft/sec or with 15 < $\bar{N} \le$ 50 or 1,000 psf $\le \bar{s}_u$ < 2,000 psf
- **Class E:** Any profile with more than 10 feet of soft clay defined as soil with plasticity index *PI* > 20, or water content w > 40 percent, and $\bar{s}_u$ < 500 psf or a soil profile with $\bar{v}_s$ < 600 ft/sec. If insufficient data are available to classify a soil profile as type A through D, a type E profile should be assumed.
- **Class F:** Soils requiring site-specific evaluations:
  – Soils vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils, quick and highly-sensitive clays, collapsible weakly-cemented soils
  – Peats and/or highly organic clays (H > 10 feet of peat and/or highly organic clay, where H = thickness of soil)
  – Very high plasticity clays (H > 25 feet with PI > 75 percent)
  – Very thick soft/medium stiff clays (H > 120 feet)

The parameters $\bar{v}_s$, $\bar{N}$, and $\bar{s}_u$ are, respectively, the average values of the shear wave velocity, Standard Penetration Test (SPT) blow count, and undrained shear strength of the upper 100 feet of soils at the site. These values may be calculated from Equation 2-6, below:

$$\bar{v}_s, \bar{N}, \bar{s}_u = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}, \frac{d_i}{N_i}, \frac{d_i}{s_{ui}}} \qquad (2\text{-}6)$$

where:

N_i = SPT blow count in soil layer "i"
n = Number of layers of similar soil materials for which data is available
d_i = Depth of layer "i"
s_{ui} = Undrained shear strength in layer "i"
v_{si} = Shear wave velocity of the soil in layer "i"

and

$$\sum_{i=1}^{n} d_i = 100\ ft \qquad (2\text{-}7)$$

Where reliable v_s data are available for the site, such data should be used to classify the site. If such data are not available, *N* data should preferably be used for cohesionless soil sites (sands, gravels), and s_u data for cohesive soil sites (clays). For rock in profile classes B and C, classification may be based either on measured or estimated values of v_s. Classification of a site as Class A rock should be based on measurements of v_s either for material at the site itself, or for similar rock materials in the vicinity; otherwise, Class B rock should be assumed. Class A or B profiles should not be assumed to be present if there is more than 10 feet of soil between the rock surface and the base of the building.

2.6.1.5 General Response Spectrum

A general, horizontal response spectrum may be constructed by plotting the following two functions in the spectral acceleration vs. structural period domain, as shown in Figure 2-1. Where a vertical response spectrum is required, it may be constructed by taking two-thirds of the spectral ordinates, at each period, obtained for the horizontal response spectrum.

$$S_a = (S_{XS}/B_S)(0.4 + 3T/T_o) \quad \text{for } 0 < T \le 0.2T_o \qquad (2\text{-}8)$$

$$S_a = (S_{X1}/(B_1 T)) \quad \text{for } T > T_o \qquad (2\text{-}9)$$

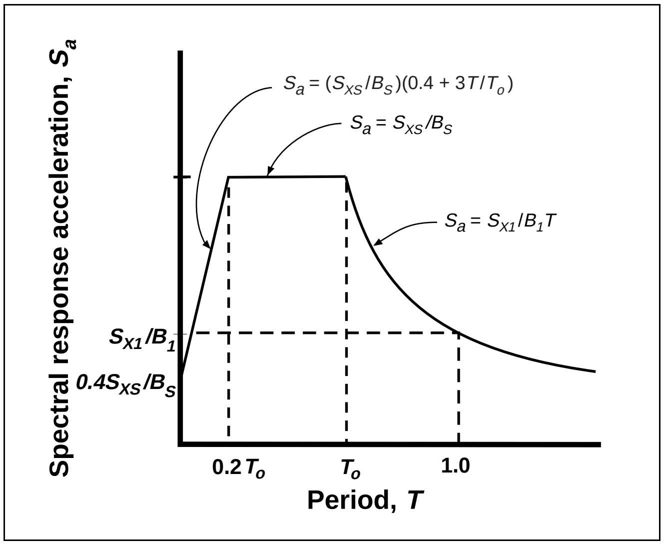

Figure 2-1 General Response Spectrum

where T_o is given by the equation

$$T_o = (S_{X1}B_S)/(S_{XS}B_1) \quad (2\text{-}10)$$

where B_S and B_1 are taken from Table 2-15.

Table 2-15 Damping Coefficients B_S and B_1 as a Function of Effective Damping β

Effective Damping β (percentage of critical)[1]	B_S	B_1
< 2	0.8	0.8
5	1.0	1.0
10	1.3	1.2
20	1.8	1.5
30	2.3	1.7
40	2.7	1.9
> 50	3.0	2.0

1. The damping coefficient should be based on linear interpolation for effective damping values other than those given.

In general, it is recommended that a 5% damped response spectrum be used for the rehabilitation design of most buildings and structural systems. Exceptions are as follows:

- For structures without exterior cladding an effective viscous damping ration, β, of 2% should be assumed.
- For structures with wood diaphragms and a large number of interior partitions and cross walls that interconnect the diaphragm levels, an effective viscous damping ratio, β, of 10% may be assumed.
- For structures rehabilitated using seismic isolation technology or enhanced energy dissipation technology, an equivalent effective viscous damping ratio, β, should be calculated using the procedures contained in Chapter 9.

In Chapter 9 of the *Guidelines*, the analytical procedures for structures rehabilitated using seismic isolation and/or energy dissipation technology make specific reference to the evaluation of earthquake demands for the BSE-2 and user-specified design earthquake hazard levels. In that chapter, the parameters: S_{aM}, S_{MS}, S_{M1}, refer respectively to the value of the spectral response acceleration parameters S_a, S_{XS}, and S_{X1}, evaluated for the BSE-2 hazard level, and the parameters S_{aD}, S_{DS}, S_{D1} in Chapter 9, refer respectively to the value of the spectral response acceleration parameters S_a, S_{XS}, and S_{X1}, evaluated for the user-specified design earthquake hazard level.

2.6.2 Site-Specific Ground Shaking Hazard

Where site-specific ground shaking characterization is used as the basis of rehabilitation design, the characterization shall be developed in accordance with this section.

2.6.2.1 Site-Specific Response Spectrum

Development of site-specific response spectra shall be based on the geologic, seismologic, and soil characteristics associated with the specific site. Response spectra should be developed for an equivalent viscous damping ratio of 5%. Additional spectra should be developed for other damping ratios appropriate to the indicated structural behavior, as discussed in Section 2.6.1.5. When the 5% damped site-specific spectrum has spectral amplitudes in the period range of greatest significance to the structural response that are less than 70 percent of the spectral amplitudes of the General Response Spectrum, an independent third-party review of the spectrum should be made by an individual with expertise in the evaluation of ground motion.

When a site-specific response spectrum has been developed and other sections of these *Guidelines* require values for the spectral response parameters, S_{XS}, S_{X1}, or T_0, they may be obtained in accordance with this section. The value of the design spectral response acceleration at short periods, S_{XS}, shall be taken as the response acceleration obtained from the site-specific spectrum at a period of 0.2 seconds, except that it should be taken as not less than 90% of the peak response acceleration at any period. In order to obtain a value for the design spectral response acceleration parameter S_{X1}, a curve of the form $S_a = S_{X1}/T$ should be graphically overlaid on the site-specific spectrum such that at any period, the value of S_a obtained from the curve is not less than 90% of that which would be obtained directly from the spectrum. The value of T_0 shall be determined in accordance with Equation 2-11. Alternatively, the values obtained in accordance with Section 2.6.1 may be used for all of these parameters.

$$T_0 = S_{X1} / S_{XS} \qquad (2\text{-}11)$$

2.6.2.2 Acceleration Time Histories

Time-History Analysis shall be performed with no fewer than three data sets (two horizontal components or, if vertical motion is to be considered, two horizontal components and one vertical component) of appropriate ground motion time histories that shall be selected and scaled from no fewer than three recorded events. Appropriate time histories shall have magnitude, fault distances, and source mechanisms that are consistent with those that control the design earthquake ground motion. Where three appropriate recorded ground-motion time history data sets are not available, appropriate simulated time history data sets may be used to make up the total number required. For each data set, the square root of the sum of the squares (SRSS) of the 5%-damped site-specific spectrum of the scaled horizontal components shall be constructed. The data sets shall be scaled such that the average value of the SRSS spectra does not fall below 1.4 times the 5%-damped spectrum for the design earthquake for periods between 0.2T seconds and 1.5T seconds (where T is the fundamental period of the building).

Where three time history data sets are used in the analysis of a structure, the maximum value of each response parameter (e.g., force in a member, displacement at a specific level) shall be used to determine design acceptability. Where seven or more time history data sets are employed, the average value of each response parameter may be used to determine design acceptability.

2.6.3 Seismicity Zones

In these *Guidelines*, seismicity zones are defined as follows.

2.6.3.1 Zones of High Seismicity

Buildings located on sites for which the 10%/50 year, design short-period response acceleration, S_{XS}, is equal to or greater than 0.5g, or for which the 10%/50 year design one-second period response acceleration, S_{X1}, is equal to or greater than 0.2g shall be considered to be located within zones of high seismicity.

2.6.3.2 Zones of Moderate Seismicity

Buildings located on sites for which the 10%/50 year, design short-period response acceleration, S_{XS}, is equal to or greater than 0.167g but is less than 0.5g, or for which the 10%/50 year, design one-second period response acceleration, S_{X1}, is equal to or greater than 0.067g but less than 0.2g shall be considered to be located within zones of moderate seismicity.

2.6.3.3 Zones of Low Seismicity

Buildings located on sites that are not located within zones of high or moderate seismicity, as defined in Sections 2.6.3.1 and 2.6.3.2, shall be considered to be located within zones of low seismicity.

2.6.4 Other Seismic Hazards

In addition to ground shaking, seismic hazards can include ground failure caused by surface fault rupture, liquefaction, lateral spreading, differential settlement, and landsliding. Earthquake-induced flooding, due to tsunami, seiche, or failure of a water-retaining structure, can also pose a hazard to a building site. The process of rehabilitating a building shall be based on the understanding that either the site is not exposed to a significant earthquake-induced flooding hazard or ground failure, or the site may be stabilized or protected from such hazards at a cost that is included along with the other rehabilitation costs. Chapter 4 describes, and provides guidance for evaluating and mitigating, these and other on-site and off-site seismic hazards.

2.7 As-Built Information

Existing building characteristics pertinent to its seismic performance—including its configuration, and the type, detailing, material strengths, and condition of the various structural and nonstructural elements, including foundations and their interconnections—shall be determined in accordance with this section. The project calculations should include documentation of these characteristics in drawings or photographs, supplemented by appropriate descriptive text. Existing characteristics of the building and site should be obtained from the following sources, as appropriate:

- Field observation of exposed conditions and configuration
- Available construction documents, engineering analyses, reports, soil borings and test logs, maintenance histories, and manufacturers' literature and test data
- Reference standards and codes from the period of construction as cited in Chapters 5 through 8
- Destructive and nondestructive examination and testing of selected building components
- Interviews with building owners, tenants, managers, the original architect and engineer, contractor(s), and the local building official

As a minimum, at least one site visit should be performed to obtain detailed information regarding building configuration and condition, site and geotechnical conditions, and any issues related to adjacent structures, and to confirm that the available construction documents are generally representative of existing conditions. If the building is a historic structure, it is also important to identify the locations of historically significant features and fabric. Care should be taken in the design and investigation process to minimize the impact of work on these features. Refer to the Secretary of the Interior's *Standards for the Treatment of Historic Properties* as discussed in Chapter 1.

2.7.1 Building Configuration

The as-built building configuration consists of the type and arrangement of existing structural elements and components composing the gravity- and lateral-load-resisting systems, and the nonstructural components. The structural elements and components shall be identified and categorized as either primary or secondary, using the criteria described in Section 2.3, with any structural deficiencies potentially affecting seismic performance also identified.

It is important, in identifying the building configuration, to account for both the intended load-resisting elements and components and the effective elements and components. The effective load-resisting systems may include building-code-conforming structural elements, nonconforming structural elements, and those nonstructural elements that actually participate in resisting gravity, lateral, or combined gravity and lateral loads, whether or not they were intended to do so by the original designers. Existing load paths should be identified, considering the effects of any modifications (e.g., additions, alterations, rehabilitation, degradation) since original construction. Potential discontinuities and weak links should also be identified, as well as irregularities that may have a detrimental effect on the building's response to lateral demands. FEMA 178 (BSSC, 1992) offers guidance for these aspects of building evaluation.

2.7.2 Component Properties

Meaningful structural analysis of a building's probable seismic behavior and reliable design of rehabilitation measures requires good understanding of the existing components (e.g., beams, columns, diaphragms), their interconnection, and their material properties (strength, deformability, and toughness). The strength and deformation capacity of existing components should be computed, as indicated in Chapters 4 through 9 and 11, based on derived material properties and detailed component knowledge. Existing component action strengths must be determined for two basic purposes: to allow calculation of their ability to deliver load to other elements and components, and to allow determination of their capacity to resist forces and deformations.

Component deformation capacity must be calculated to allow validation of overall element and building deformations and their acceptability for the selected Rehabilitation Objectives. In general, component capacities are calculated as "expected values" that account for the mean material strengths as well as the probable effects of strain hardening and/or degradation. The exception to this is the calculation of strengths used to evaluate the adequacy of component force actions with little inherent ductility (force-controlled behaviors). For these evaluations, lower-bound strength estimates—taking into account the possible variation in material strengths—are used for determination of capacity. Guidance on how to obtain these expected and lower-bound values is provided in Chapters 5 through 8 for the commonly used structural materials and systems.

Knowledge of existing component configuration, quality of construction, physical condition, and interconnection to other structural components is necessary to compute strength and deformation capacities. This knowledge should be obtained by visual surveys of condition, destructive and nondestructive testing, and field measurement of dimensions, as appropriate. Even with an exhaustive effort to maximize knowledge, uncertainty will remain regarding the validity of computed component strength and deformation capacities. To account for this uncertainty, a knowledge factor, κ, is utilized in the capacity evaluations. Two possible values exist for κ, based on the reliability of available knowledge—classified as either minimum or comprehensive.

When only a minimum level of knowledge is available, a κ value of 0.75 shall be included in component capacity and deformation analyses. The following characteristics represent the minimum appropriate level of effort in gaining knowledge of structural configuration:

- Records of the original construction and any modifications, including structural and architectural drawings, are generally available. In the absence of structural drawings, a set of record drawings and/or sketches is prepared, documenting both gravity and lateral systems.
- A visual condition survey is performed on the accessible primary elements and components, with verification that the size, location, and connection of these elements is as indicated on the available documentation.
- A limited program of in-place testing is performed, as indicated in Chapters 5 through 8, to quantify the material properties, component condition, and dimensions of representative primary elements with quantification of the effects of any observable deterioration. Alternatively, default values provided in Chapters 5 through 8 are utilized for material strengths, taking into account the observed condition of these materials; if significant variation is found in the condition or as-tested properties of materials, consideration should be given to grouping those components with similar condition or properties so that the coefficient of variation within a group does not exceed 30%.
- Knowledge of any site-related concerns—such as pounding from neighboring structures, party wall effects, and soil or geological problems including risks of liquefaction—has been gained through field surveys and research.
- Specific foundation- and material-related concerns cited in Chapters 4 through 8, as applicable, have been examined, and knowledge of their influence on building performance has been gained.

A κ value of 1.0 may be used where comprehensive knowledge and understanding of component configuration has been obtained. Comprehensive knowledge may be assumed when all of the following factors exist:

- Original construction records, including drawings and specifications, as well as any post-construction modification data, are available and explicitly depict as-built conditions. Where such documents are not available, drawings and sketches are developed based on detailed surveys of the primary structural elements. Such surveys include destructive and/or nondestructive investigation as required to determine the size, number, placement, and type of obscured items such as bolts and reinforcing bars. In addition, documentation is developed for representative secondary elements.
- Extensive in-place testing is performed as indicated in Chapters 4 through 8 to quantify material properties, and component conditions and dimensions or records of the results of quality assurance tests constructed during testing are available. Coefficients of variability for material strength test results are less than 20%, or components are grouped and additional testing is performed such that the material strength test results for each group have coefficients of variation within this limit.
- Knowledge of any site-related concerns—such as pounding from neighboring structures, party wall effects, and soil or geological problems including

risks of liquefaction—has been gained from thorough visual survey and research efforts.

- Specific foundation- and material-related concerns cited in Chapters 4 through 8, as applicable, have been examined and knowledge of their influence on building performance has been gained.

Whenever practical, investigation should be sufficiently thorough to allow the use of a single value of κ for all building components and elements. If extenuating circumstances prevent use of a common κ value for certain components, multiple κ values should be used in the analysis, as appropriate to the available knowledge of the individual components. When a nonlinear analysis procedure is employed, the level of investigation should be sufficient to allow comprehensive knowledge of the structure ($\kappa = 1.0$).

2.7.3 Site Characterization and Geotechnical Information

Data on surface and subsurface conditions at the site, including the configuration of foundations, shall be obtained for use in building analyses. Data shall be obtained from existing documents, visual site reconnaissance, or a program of subsurface investigation. If adequate geotechnical data are not available from previous investigations, a program of site-specific subsurface investigation should be considered for sites in areas subject to liquefaction, lateral spreading, or landsliding, and for all buildings with an Enhanced Rehabilitation Objective. Additional guidelines for site characterization and subsurface investigation are contained in Chapter 4.

A site reconnaissance should always be performed. In the course of this reconnaissance, variances from the building drawings should be noted. Such variances could include foundation modifications that are not shown on the existing documentation. Off-site development that should be noted could include buildings or grading activities that may impose a load or reduce the level of lateral support to the structure. Indicators of poor foundation performance—such as settlements of floor slabs, foundations or sidewalks, suggesting distress that could affect building performance during a future earthquake—should be noted.

2.7.4 Adjacent Buildings

Data should be collected on the configuration of adjacent structures when such structures have the potential to influence the seismic performance of the rehabilitated building. Data collected should be sufficient to permit analysis of the potential interaction issues identified below, as applicable. In some cases, it may not be possible to obtain adequate information on adjacent structures to permit a meaningful evaluation. In such cases, the owner should be notified of the potential consequences of these interactions.

2.7.4.1 Building Pounding

Data on adjacent structures should be collected to permit investigation of the potential effects of building pounding whenever the side of the adjacent structure is located closer to the building than 4% of the building height above grade at the location of potential impacts.

Building pounding can alter the basic response of the building to ground motion, and impart additional inertial loads and energy to the building from the adjacent structure. Of particular concern is the potential for extreme local damage to structural elements at the zones of impact. (See Section 2.11.10.)

2.7.4.2 Shared Element Condition

Data should be collected on all adjacent structures that share elements in common with the building. Buildings sharing common elements, such as party walls, have several potential problems. If the buildings attempt to move independently, one building may pull the shared element away from the other, resulting in a partial collapse. If the buildings behave as an integral unit, the additional mass and inertial loads of one structure may result in extreme demands on the lateral-force-resisting system of the other. (See Section 2.11.9.)

2.7.4.3 Hazards from Adjacent Structure

Data should be collected on all structures that have the potential to damage the building with falling debris, or other earthquake-induced physical hazards such as aggressive chemical leakage, fire, or explosion.

Consideration should be given to hardening those portions of the building that may be impacted by debris or other hazards from adjacent structures. Where Immediate Occupancy of the building is desired, and ingress to the building may be impaired by such hazards, consideration should be given to providing for suitably resistant access to the building. Sufficient information must be collected on adjacent structures to allow preliminary evaluation of the likelihood and nature of hazards such as potential falling debris, fire, and blast pressures. Evaluations similar to those in FEMA 154, *Rapid Visual Screening of Buildings for Seismic Hazards: A Handbook* (ATC, 1988), should be adequate for this purpose.

2.8 Rehabilitation Methods

The scope of building structural alterations and modifications required to meet the selected Rehabilitation Objective shall be determined in accordance with one of the methods described in this section. In addition, rehabilitation of historic buildings should be carefully considered in accordance with the discussion in Chapter 1.

2.8.1 Simplified Method

The Simplified Method allows for design of building rehabilitation measures without requiring analyses of the entire building's response to earthquake hazards. This method is not applicable to all buildings and can be used only to achieve Limited Rehabilitation Objectives (Section 2.4.3).

The Simplified Method may be used to achieve a Rehabilitation Objective consisting of the Life Safety Performance Level (3-C) for a BSE-1 earthquake for buildings meeting all of the following conditions:

- The building conforms to one of the Model Building Types indicated in Table 10-1, as well as all limitations indicated in that table with regard to number of stories, regularity, and seismic zone; and

- A complete evaluation of the building is performed in accordance with FEMA 178 (BSSC, 1992), and all deficiencies identified in that evaluation are addressed by the selected Simplified Rehabilitation Methods.

Any building may be partially rehabilitated to achieve a Limited Rehabilitation Objective using the Simplified Method, subject to the limitations of Section 2.4.3.

The Simplified Method may not be used for buildings intended to meet the BSO or any Enhanced Rehabilitation Objectives. For those buildings and other buildings not meeting the limitations for the Simplified Method, the Systematic Method shall be used.

2.8.2 Systematic Method

Rehabilitation programs for buildings and objectives that do not qualify for Simplified Rehabilitation under Section 2.8.1 shall be designed in accordance with this section. The basic approach shall include the following:

- The structure shall be analyzed to determine if it is adequate to meet the selected Rehabilitation Objective(s) and, if it is not adequate, to identify specific deficiencies. If initial analyses indicate that key elements or components of the structure do not meet the acceptance criteria, it may be possible to demonstrate acceptability by using more detailed and accurate analytical procedures. Section 2.9 provides information on alternative analytical procedures that may be used.

- One or more rehabilitation strategies shall be developed to address the deficiencies identified in the preliminary evaluation. Alternative rehabilitation strategies are presented in Section 2.10.

- A preliminary rehabilitation design shall be developed that is consistent with the rehabilitation strategy.

- The structure and the preliminary rehabilitation measures shall be analyzed to determine whether the rehabilitated structure will be adequate to meet the selected Rehabilitation Objective(s).

- The process shall be repeated as required until a design solution is obtained that meets the selected Rehabilitation Objective(s), as determined by the analysis.

2.9 Analysis Procedures

An analysis of the structure shall be conducted to determine the distribution of forces and deformations induced in the structure by the design ground shaking and other seismic hazards corresponding with the selected Rehabilitation Objective(s). The analysis shall address the seismic demands and the capacity to resist these demands for all elements in the structure that either:

- Are essential to the lateral stability of the structure (primary elements); or
- Are essential to the vertical load-carrying integrity of the building; or
- Are otherwise critical to meeting the Rehabilitation Objective and could be subject to damage as a result of the building's response to the earthquake hazards.

The analysis procedure shall consist of one of the following:

- Linear analysis, in accordance with Section 3.3, including Linear Static Procedure (LSP) (see Section 3.3.1), and Linear Dynamic Procedure (LDP) (see Section 3.3.2), including:
  – Response Spectrum Analysis (see Section 3.3.2.2C), and
  – Linear Time-History Analysis (see Section 3.3.2.2D), or
- Nonlinear analysis, in accordance with Section 3.3, including Nonlinear Static Procedure (NSP) in Section 3.3.3 and Nonlinear Dynamic Procedure (NDP) in Section 3.3.4, or
- Alternative rational analysis

Limitations with regard to the use of these procedures are given in Sections 2.9.1, 2.9.2, and 2.9.3. Criteria used to determine whether the results of an analysis indicate acceptable performance for the building are discussed in Section 2.9.4.

2.9.1 Linear Procedures

Linear procedures may be used for any of the rehabilitation strategies contained in Section 2.10 except those strategies incorporating the use of supplemental energy dissipation systems and some types of seismic isolation systems. For the specific analysis procedures applicable to these rehabilitation strategies, refer to Chapter 9.

The results of the linear procedures can be very inaccurate when applied to buildings with highly irregular structural systems, unless the building is capable of responding to the design earthquake(s) in a nearly elastic manner. Therefore, linear procedures should not be used for highly irregular buildings, unless the earthquake ductility demands on the building are suitably low.

2.9.1.1 Method to Determine Applicability of Linear Procedures

The methodology indicated in this section may be used to determine whether a building can be analyzed with sufficient accuracy by linear procedures. The basic approach is to perform a linear analysis using the loads defined in either Section 3.3.1 or 3.3.2 and then to examine the results of this analysis to identify the magnitude and uniformity of distribution of inelastic demands on the various components of the primary lateral-force-resisting elements. The magnitude and distribution of inelastic demands are indicated by demand-capacity ratios (DCRs), as defined below. Note that these DCRs are not used to determine the acceptability of component behavior. The adequacy of structural components and elements must be evaluated using the procedures contained in Chapter 3, together with the acceptance criteria provide in Chapters 4 through 8. DCRs are used only to determine a structure's regularity. It should be noted that for complex structures, such as buildings with perforated shear walls, it may be easier to use one of the nonlinear procedures than to ensure that the building has sufficient regularity to permit use of linear procedures.

DCRs for existing and added building components shall be computed in accordance with the equation:

$$DCR = \frac{Q_{UD}}{Q_{CE}} \tag{2-12}$$

where:

Q_{UD} = Force calculated in accordance with Section 3.4, due to the gravity and earthquake loads of Section 3.3

Q_{CE} = Expected strength of the component or element, calculated in accordance with Chapters 5 through 8

DCRs should be calculated for each controlling action (such as axial force, moment, shear) of each component. If all of the computed controlling DCRs for a component are less than or equal to 1.0, then the component is expected to respond elastically to the earthquake ground shaking being evaluated. If one or more of the computed DCRs for a component are greater than 1.0, then the component is expected to respond inelastically to the earthquake ground shaking. The largest DCR calculated for a given component defines the critical action for the component, i.e., the mode in which the component will first yield, or fail. This DCR is termed the critical component DCR. If an element is composed of multiple components, then the component with the largest computed DCR is the critical component for the element, i.e., this will be the first component in the element to yield, or fail. The largest DCR for any component in an element at a particular story is termed the critical element DCR at that story.

If the DCRs computed for all of the critical actions (axial force, moment, shear) of all of the components (such as beams, columns, wall piers, braces, and connections) of the primary elements are less than 2.0, then linear procedures are applicable, regardless of considerations of regularity. If some computed DCRs exceed 2.0, then linear procedures should not be used if any of the following apply:

- There is an in-plane discontinuity in any primary element of the lateral-force-resisting system. In-plane discontinuities occur whenever a lateral-force-resisting element is present in one story, but does not continue, or is offset, in the story immediately below. Figure 2-2 depicts such a condition. This limitation need not apply if the coefficient J in Equation 3-15 is taken as 1.0.

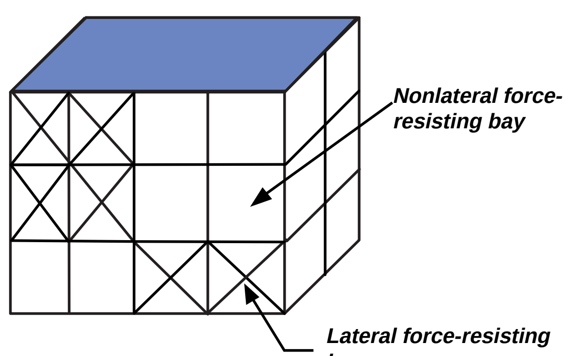

Figure 2-2 In-Plane Discontinuity in Lateral System

- There is an out-of-plane discontinuity in any primary element of the lateral-force-resisting system. An out-of-plane discontinuity exists when an element in one story is offset relative to the continuation of that element in an adjacent story, as depicted in Figure 2-3. This limitation need not apply if the coefficient J in Equation 3-15 is taken as 1.0.

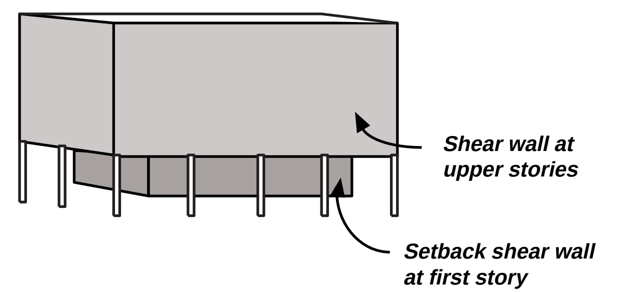

Figure 2-3 Typical Building with Out-of-Plane Offset Irregularity

- There is a severe weak story irregularity present at any story in any direction of the building. A severe weak story irregularity may be deemed to exist if the ratio of the average shear DCR for any story to that for an adjacent story in the same direction exceeds 125%. The average DCR for a story may be calculated by the equation:

$$\overline{DCR} = \frac{\sum_{1}^{n} DCR_i V_i}{\sum_{1}^{n} V_i} \tag{2-13}$$

where:

$\overline{DCR}$	=	Average DCR for the story
DCR_i	=	Critical action DCR for element i
V_i	=	Total calculated lateral shear force in an element i due to earthquake response, assuming that the structure remains elastic
n	=	Total number of elements in the story

For buildings with flexible diaphragms, each line of framing should be independently evaluated.

- There is a severe torsional strength irregularity present in any story. A severe torsional strength irregularity may be deemed to exist in a story when the diaphragm above the story is not flexible and the ratio of the critical element DCRs for primary elements on one side of the center of resistance in a given direction for a story, to those on the other side of the center of resistance for the story, exceeds 1.5.

If the guidelines above indicate that a linear procedure is applicable, then either the LSP or the LDP may be used, unless one or more of the following apply, in which case the LSP should not be used:

- The building height exceeds 100 feet.
- The ratio of the building's horizontal dimension at any story to the corresponding dimension at an adjacent story exceeds 1.4 (excluding penthouses).
- The building is found to have a severe torsional stiffness irregularity in any story. A severe torsional stiffness irregularity may be deemed to exist in a story if the diaphragm above the story is not flexible and the results of the analysis indicate that the drift along any side of the structure is more than 150% of the average story drift.
- The building is found to have a severe vertical mass or stiffness irregularity. A severe vertical mass or stiffness irregularity may be deemed to exist when the average drift in any story (except penthouses) exceeds that of the story above or below by more than 150%.
- The building has a nonorthogonal lateral-force-resisting system.

2.9.2 Nonlinear Procedures

Nonlinear Analysis Procedures may be used for any of the rehabilitation strategies contained in Section 2.10. Nonlinear procedures are especially recommended for analysis of buildings having irregularities as identified in Section 2.9.1.1. The NSP is mainly suitable for buildings without significant higher-mode response. The NDP is suitable for any structure, subject to the limitations in Section 2.9.2.2.

2.9.2.1 Nonlinear Static Procedure (NSP)

The NSP may be used for any structure and any Rehabilitation Objective, with the following exceptions and limitations.

- The NSP should not be used for structures in which higher mode effects are significant, unless an LDP evaluation is also performed. To determine if higher modes are significant, a modal response spectrum analysis should be performed for the structure using sufficient modes to capture 90% mass participation, and a second response spectrum analysis should be performed considering only the first mode participation. Higher mode effects should be considered significant if the shear in any story calculated from the modal analysis considering all modes required to obtain 90% mass participation exceeds 130% of the corresponding story shear resulting from the analysis considering only the first mode response. When an LDP is performed to supplement an NSP for a structure with significant higher mode effects, the acceptance criteria values for deformation-controlled actions (m values), provided in Chapters 5 through 9, may be increased by a factor of 1.33.
- The NSP should not be used unless comprehensive knowledge of the structure has been obtained, as indicated in Section 2.7.2.

2.9.2.2 Nonlinear Dynamic Procedure (NDP)

The NDP may be used for any structure and any Rehabilitation Objective, with the following exceptions and limitations.

- The NDP is not recommended for use with wood frame structures.
- The NDP should not be utilized unless comprehensive knowledge of the structure has been obtained, as indicated in Section 2.7.2.
- The analysis and design should be subject to review by an independent third-party professional engineer with substantial experience in seismic design and nonlinear procedures.

2.9.3 Alternative Rational Analysis

Nothing in the *Guidelines* should be interpreted as preventing the use of any alternative analysis procedure that is rational and based on fundamental principles of engineering mechanics and dynamics. Such alternative analyses should not adopt the acceptance criteria contained in the *Guidelines* without careful review as to their applicability. All projects using alternative rational analysis procedures should be subject to review by an independent third-party professional engineer with substantial experience in seismic design.

2.9.4 Acceptance Criteria

The Analysis Procedures indicate the building's response to the design earthquake(s) and the forces and deformations imposed on the various components, as well as global drift demands on the structure. When LSP or LDP analysis is performed, acceptability of component behavior is evaluated for each of the component's various actions using Equation 3-18 for ductile (deformation-controlled) actions and Equation 3-19 for nonductile (force-controlled) actions. Figure 2-4 indicates typical idealized force-deformation curves for various types of component actions.

The type 1 curve is representative of typical ductile behavior. It is characterized by an elastic range (point 0 to point 1 on the curve), followed by a plastic range (points 1 to 3) that may include strain hardening or softening (points 1 to 2), and a strength-degraded range (points 2 to 3) in which the residual force that can be resisted is significantly less than the peak strength, but still substantial. Acceptance criteria for primary elements that exhibit this behavior are typically within the elastic or plastic ranges between points 1 and 2, depending on the Performance Level. Acceptance criteria for secondary elements can be within any of the ranges. Primary component actions exhibiting this behavior are considered deformation-controlled if the strain-hardening or strain-softening range is sufficiently large $e > 2g$; otherwise, they are considered force-controlled. Secondary component actions exhibiting this behavior are typically considered to be deformation-controlled.

The type 2 curve is representative of another type of ductile behavior. It is characterized by an elastic range and a plastic range, followed by a rapid and complete loss of strength. If the plastic range is sufficiently large ($e \geq 2g$), this behavior is categorized as deformation-controlled. Otherwise it is categorized as force-controlled. Acceptance criteria for primary and secondary components exhibiting this behavior will be within the elastic or plastic ranges, depending on the performance level.

The type 3 curve is representative of a brittle or nonductile behavior. It is characterized by an elastic range, followed by a rapid and complete loss of strength. Component actions displaying this behavior are always categorized as force-controlled. Acceptance criteria for primary and secondary components exhibiting this behavior are always within the elastic range.

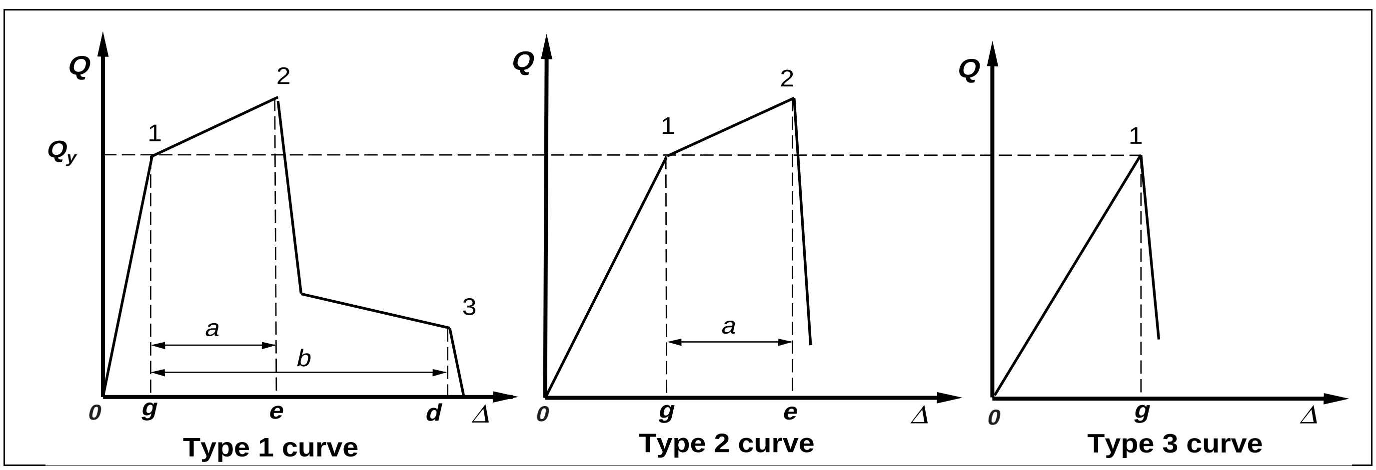

Figure 2-4 General Component Behavior Curves

Figure 2-5 shows an idealized force versus deformation curve that is used throughout the *Guidelines* to specify acceptance criteria for deformation-controlled component and element actions for any of the four basic types of materials. Linear response is depicted between point *A* (unloaded component) and an effective yield point *B*. The slope from *B* to *C* is typically a small percentage (0–10%) of the elastic slope, and is included to represent phenomena such as strain hardening. *C* has an ordinate that represents the strength of the component, and an abscissa value equal to the deformation at which significant strength degradation begins (line *CD*). Beyond point *D*, the component responds with substantially reduced strength to point *E*. At deformations greater than point *E*, the component strength is essentially zero.

In Figure 2-4, Q_y represents the yield strength of the component. In a real structure, the yield strength of individual elements that appear similar will actually have some variation. This is due to inherent variability in the material strength comprising the individual elements as well as differences in workmanship and physical condition. When evaluating the behavior of deformation-controlled components, the expected strength, Q_{CE}, rather than the yield strength Q_y is used. Q_{CE} is defined as the mean value of resistance at the deformation level anticipated, and includes consideration of the variability discussed above as well as phenomena such as strain hardening and plastic section development. When evaluating the behavior of force-controlled components, a lower bound estimate of the component strength, Q_{CL}, is considered. Q_{CL} is statistically defined as the mean minus one standard deviation of the yield strengths Q_y for a population of similar components.

For some components it is convenient to prescribe acceptance criteria in terms of deformation (e.g., θ or Δ), while for others it is more convenient to give criteria in terms of deformation ratios. To accommodate this, two types of idealized force versus deformation curves are used in the *Guidelines* as illustrated in Figures 2-5(a) and (b). Figure 2-5(a) shows normalized force (Q/Q_{CE}) versus deformation (θ or Δ) and the parameters *a*, *b*, and *c*. Figure 2-5(b) shows normalized force (Q/Q_{CE}) versus deformation ratio (θ/θ_y, Δ/Δ_y, or Δ/h) and the parameters *d*, *e*, and *c*. Elastic stiffnesses and values for the parameters *a*, *b*, *c*, *d*, and *e* that can be used for modeling components are given in Chapters 5 through 8.

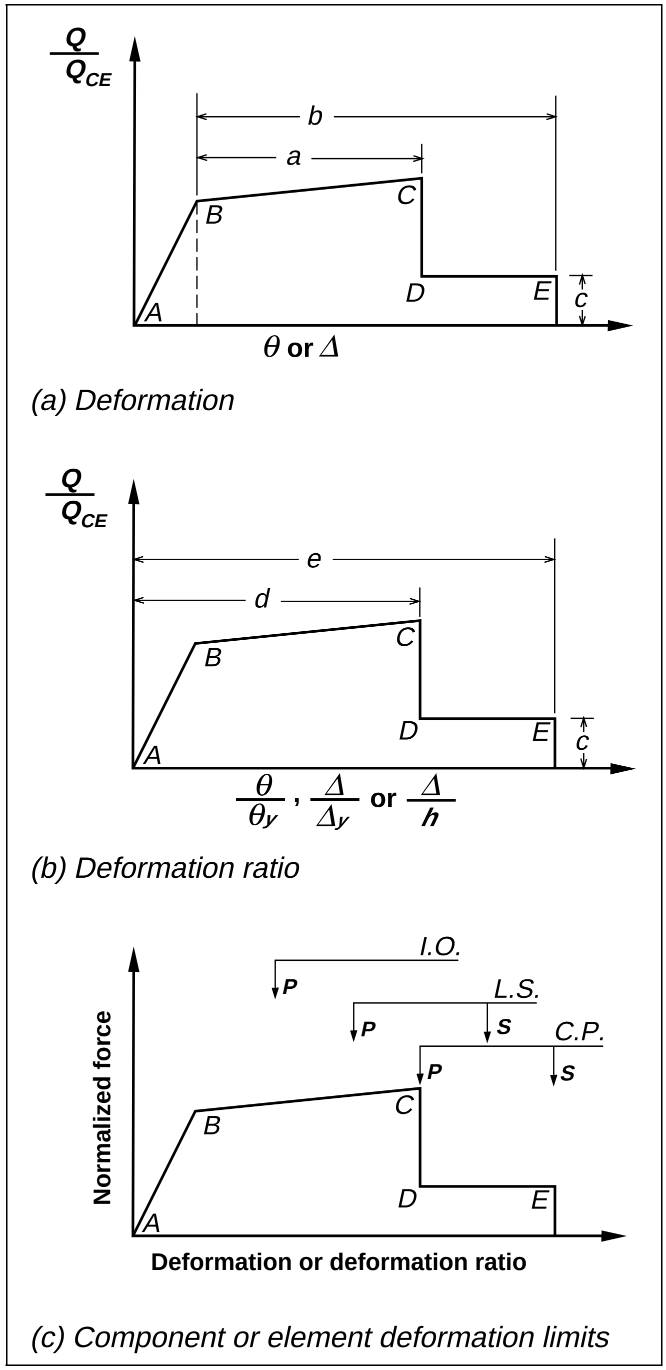

Figure 2-5 ***Idealized Component Load versus Deformation Curves for Depicting Component Modeling and Acceptability***

Figure 2-5(c) graphically shows the approximate deformation or deformation ratio, in relation to the idealized force versus deformation curve, that are deemed acceptable in the *Guidelines* for Primary (*P*) and Secondary (*S*) components for Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP) Performance Levels. Numerical values of the acceptable deformations or deformation ratios are given in Chapters 5 through 8 for all types of components and elements.

If nonlinear procedures are used, component capacities consist of permissible inelastic deformation demands for deformation-controlled components, and of permissible strength demands for force-controlled components. If linear procedures are used, capacities are defined as the product of factors m and expected strengths Q_{CE} for deformation-controlled components and as permissible strength demands for force-controlled components. Tables 2-16 and 2-17 summarize these capacities. In this table, κ is the knowledge-based factor defined in Section 2.7.2, and σ is the standard deviation of the material strengths. Detailed guidelines on the calculation of individual component force and deformation capacities may be found in the individual materials chapters as follows:

- Foundations—Chapter 4
- Elements and components composed of steel or cast iron—Chapter 5
- Elements and components composed of reinforced concrete—Chapter 6
- Elements and components composed of reinforced or unreinforced masonry—Chapter 7
- Elements and components composed of timber, light metal studs, gypsum, or plaster products—Chapter 8
- Seismic isolation systems and energy dissipation systems—Chapter 9
- Nonstructural (architectural, mechanical, and electrical) components—Chapter 11
- Elements and components comprising combinations of materials—covered in the chapters associated

Table 2-16 Calculation of Component Action Capacity—Linear Procedures

Parameter	Deformation-Controlled	Force-Controlled
Existing Material Strength	Expected mean value with allowance for strain hardening	Lower bound value (approximately -1σ level)
Existing Action Capacity	$\kappa \cdot Q_{CE}$	$\kappa \cdot Q_{CE}$
New Material Strength	Expected material strength	Specified material strength
New Action Capacity	Q_{CE}	Q_{CE}

Note: Capacity reduction (ϕ) factors are typically taken as unity in the evaluation of capacities.

Table 2-17 Calculation of Component Action Capacity—Nonlinear Procedures

Parameter	Deformation-Controlled	Force-Controlled
Deformation Capacity—Existing Component	$\kappa \cdot$ deformation limit	N/A
Deformation Capacity—New Component	deformation limit	N/A
Strength Capacity—Existing Component	N/A	$\kappa \cdot Q_{CL}$
Strength Capacity—New Element	N/A	Q_{CL}

Note: Capacity reduction (ϕ) factors are typically taken as unity in the evaluation of capacities.

Acceptance criteria for elements and components for which criteria are not presented in the *Guidelines* shall be determined by a qualification testing program, in accordance with the procedures of Section 2.13.

2.10 Rehabilitation Strategies

Rehabilitation of buildings may be achieved by one or more of the strategies indicated in this section. Although not specifically required by any of the strategies, it is very beneficial for the rehabilitated building's lateral-force-resisting system to have an appropriate level of redundancy, so that any localized failure of a few elements of the system will not result in local collapse or an instability. This should be considered when developing rehabilitation designs.

2.10.1 Local Modification of Components

Some existing buildings have substantial strength and stiffness; however, some of their components do not have adequate strength, toughness, or deformation capacity to satisfy the Rehabilitation Objectives. An appropriate strategy for such structures may be to perform local modifications of those components that are inadequate, while retaining the basic configuration of the building's lateral-force-resisting system. Local modifications that can be considered include improvement of component connectivity, component strength, and/or component deformation capacity. This strategy tends to be the most economical approach to rehabilitation when only a few of the building's components are inadequate.

Local strengthening allows one or more understrength elements or connections to resist the strength demands predicted by the analysis, without affecting the overall response of the structure. This could include measures such as cover plating steel beams or columns, or adding plywood sheathing to an existing timber diaphragm. Such measures increase the strength of the element or component and allow it to resist more earthquake-induced force before the onset of damage.

Local corrective measures that improve the deformation capacity or ductility of a component allow it to resist large deformation levels with reduced amounts of damage, without necessarily increasing the strength. One such measure is placement of a confinement jacket around a reinforced concrete column to improve its ability to deform without spalling or degrading reinforcement splices. Another measure is reduction of the cross section of selected structural components to increase their flexibility and response displacement capacity.

2.10.2 Removal or Lessening of Existing Irregularities and Discontinuities

Stiffness, mass, and strength irregularities are common causes of undesirable earthquake performance. When reviewing the results of a linear analysis, the irregularities can be detected by examining the distribution of structural displacements and DCRs. When reviewing the results of a nonlinear analysis, the irregularities can be detected by examining the distribution of structural displacements and inelastic deformation demands. If the values of structural displacements, DCRs, or inelastic deformation demands predicted by the analysis are unbalanced, with large concentrations of high values within one story or at one side of a building, then an irregularity exists. Such irregularities are often, but not always, caused by the presence of a discontinuity in the structure, as for example, termination of a perimeter shear wall above the first story. Simple removal of the irregularity may be sufficient to reduce demands predicted by the analysis to acceptable levels. However, removal of discontinuities may be inappropriate in the case of historic buildings, and the effect of such alterations on important historic features should be considered carefully.

Effective corrective measures for removal or reduction of irregularities and discontinuities, such as soft or weak stories, include the addition of braced frames or shear walls within the soft/weak story. Torsional irregularities can be corrected by the addition of moment frames, braced frames, or shear walls to balance the distribution of stiffness and mass within a story. Discontinuous components such as columns or walls can be extended through the zone of discontinuity.

Partial demolition can also be an effective corrective measure for irregularities, although this obviously has significant impact on the appearance and utility of the building, and this may not be an appropriate alternative for historic structures. Portions of the structure that create the irregularity, such as setback towers or side wings, can be removed. Expansion joints can be created to transform a single irregular building into multiple regular structures; however, care must be taken to avoid the potential problems associated with pounding.

2.10.3 Global Structural Stiffening

Some flexible structures behave poorly in earthquakes because critical components and elements do not have adequate ductility or toughness to resist the large lateral deformations that ground shaking induces in the structure. For structures comprising many such elements, an effective way to improve performance is to stiffen the structure so that its response produces less lateral deformation. Construction of new braced frames or shear walls within an existing structure are effective measures for adding stiffness.

2.10.4 Global Structural Strengthening

Some existing buildings have inadequate strength to resist lateral forces. Such structures exhibit inelastic behavior at very low levels of ground shaking. Analyses of such buildings indicate large DCRs (or inelastic deformation demands) throughout the structure. By providing supplemental strength to such a building's lateral-force-resisting system, it is possible to raise the threshold of ground motion at which the onset of damage occurs. Shear walls and braced frames are effective elements for this purpose; however, they may be significantly stiffer than the structure to which they are added, requiring that they be designed to provide nearly all of the structure's lateral resistance. Moment-resisting frames, being more flexible, may be more compatible with existing elements in some structures; however, such flexible elements may not become effective in the building's response until existing brittle elements have already been damaged.

2.10.5 Mass Reduction

Two of the primary characteristics that control the amount of force and deformation induced in a structure by ground motion are its stiffness and mass. Reductions in mass result in direct reductions in both the amount of force and deformation demand produced by earthquakes, and therefore can be used in lieu of structural strengthening and stiffening. Mass can be reduced through demolition of upper stories, replacement of heavy cladding and interior partitions, or removal of heavy storage and equipment loads.

2.10.6 Seismic Isolation

When a structure is seismically isolated, compliant bearings are inserted between the superstructure and its foundations. This produces a system (structure and isolation bearings) with fundamental response that consists of nearly rigid body translation of the structure above the bearings. Most of the deformation induced in the isolated system by the ground motion occurs within the compliant bearings, which have been specifically designed to resist these concentrated displacements. Most bearings also have excellent energy dissipation characteristics (damping). Together, this results in greatly reduced demands on the existing elements of the structure, including contents and nonstructural components. For this reason, seismic isolation is often an appropriate strategy to achieve Enhanced Rehabilitation Objectives that include protection of historic fabric, valuable contents, and equipment, or for buildings that contain important operations and functions. This technique is most effective for relatively stiff buildings with low profiles and large mass. It is less effective for light, flexible structures.

2.10.7 Supplemental Energy Dissipation

A number of technologies are available that allow the energy imparted to a structure by ground motion to be dissipated in a controlled manner through the action of special devices—such as fluid viscous dampers (hydraulic cylinders), yielding plates, or friction pads—resulting in an overall reduction in the displacements of the structure. The most common devices dissipate energy through frictional, hysteretic, or viscoelastic processes. In order to dissipate substantial energy, dissipation devices must typically undergo significant deformation (or stroke) which requires that the structural experience substantial lateral displacements. Therefore, these systems are most effective in structures that are relatively flexible and have some inelastic deformation capacity. Energy dissipaters are most commonly installed in structures as components of braced frames. Depending on the characteristics of the device, either static or dynamic stiffness is added to the structure as well as energy dissipation capacity (damping). In some cases, although the structural displacements are reduced, the forces delivered to the structure can actually be increased.

2.11 General Analysis and Design Requirements

The detailed guidelines of this section apply to all buildings rehabilitated to achieve either the BSO or any Enhanced Rehabilitation Objectives. Though compliance with the guidelines in this section is not required for buildings rehabilitated to Limited Rehabilitation Objectives, such compliance should be considered. Unless otherwise noted, all numerical values apply to the Life Safety Performance Level, and must be multiplied by 1.25 to apply to Immediate Occupancy.

2.11.1 Directional Effects

The lateral-load-resisting system shall be demonstrated to be capable of responding to ground-motion-producing lateral forces in any horizontal direction. For buildings with orthogonal primary axes of resistance, this may be satisfied by evaluating the response of the structure to such forces in each of the two orthogonal directions. As a minimum, the effects of structural response in each of these orthogonal directions shall be considered independently. In addition, the combined effect of simultaneous response in both directions shall be considered, in accordance with the applicable procedures of Section 3.2.7.

2.11.2 P-Δ Effects

The structure shall be investigated to ensure that lateral drifts induced by earthquake response do not result in a condition of instability under gravity loads. At each story, the quantity θ_i shall be calculated for each direction of response, as follows:

$$\theta_i = \frac{P_i \delta_i}{V_i h_i} \qquad (2\text{-}14)$$

where:

P_i = Portion of the total weight of the structure including dead, permanent live, and 25% of transient live loads acting on the columns and bearing walls within story level *i*

V_i = Total calculated lateral shear force in the direction under consideration at story *i* due to earthquake response, assuming that the structure remains elastic

h_i = Height of story *i*, which may be taken as the distance between the centerline of floor framing at each of the levels above and below, the distance between the top of floor slabs at each of the levels above and below, or similar common points of reference

δ_i = Lateral drift in story *i*, in the direction under consideration, at its center of rigidity, using the same units as for measuring h_i

In any story in which θ_i is less than or equal to 0.1, the structure need not be investigated further for stability concerns. When the quantity θ_i in a story exceeds 0.1, the analysis of the structure shall consider P-Δ effects, in accordance with the applicable procedures of Section 3.2.5. When the value of θ_i exceeds 0.33, the structure should be considered potentially unstable and the rehabilitation design modified to reduce the computed lateral deflections in the story.

2.11.3 Torsion

Analytical models used to evaluate the response of the building to earthquake ground motion shall account for the effects of torsional response resulting from differences in the plan location of the center of mass and center of rigidity of the structure at all diaphragm levels that are not flexible.

2.11.4 Overturning

The effects of overturning at each level of the structure shall be evaluated cumulatively from the top of the structure to its base (See the commentary and further guidance in the sidebar, “Overturning Issues and Alternative Methods.”)

2.11.4.1 Linear Procedures

When a linear procedure is followed, each primary element at each level of the structure shall be investigated for stability against overturning under the effects of seismic forces applied at and above the level under consideration. Overturning effects may be resisted either through the stabilizing effect of dead loads or through positive connection of the element to structural components located below.

Where dead loads are used to resist the effects of overturning, the following shall be satisfied:

$$M_{ST} > M_{OT} / (C_1 C_2 C_3 J) \qquad (2\text{-}15)$$

where

M_{OT} = Total overturning moment induced on the element by seismic forces applied at and above the level under consideration

M_{ST} = Stabilizing moment produced by dead loads acting on the element, calculated as the sum of the products of each separate dead load and the horizontal distance between its vertical line of action and the centroid of the resisting

Overturning Issues and Alternative Methods

Response to earthquake ground motion results in a tendency for structures, and individual vertical elements of structures, to overturn about their bases. Although actual overturning is very rare, overturning effects can result in significant stresses, which have caused some local and global failures. In new building design, earthquake effects, including overturning, are evaluated for lateral forces that are significantly reduced (by the *R*-factor) from those which may actually develop in the structure.

For elements with positive attachment between levels, that behave as single units, such as reinforced concrete walls, the overturning effects are resolved into component forces (e.g., flexure and shear at the base of the wall) and the element is then proportioned with adequate strength to resist these overturning effects resulting from the reduced force levels.

Some elements, such as wood shear walls and foundations, may not be provided with positive attachment between levels. For them, an overturning stability check is performed. If the element has sufficient dead load to remain stable under the overturning effects of the design lateral forces and sufficient shear connection to the level below, then the design is deemed adequate. However, if dead load is inadequate to provide stability, then hold-downs, piles, or other types of uplift anchors are provided to resist the residual overturning caused by the design forces.

In the linear and nonlinear procedures of the *Guidelines*, lateral forces are not reduced by an *R*-factor, as they are for new buildings. Thus, computed overturning effects are larger than typically calculated for new buildings. Though the procedure used for new buildings is not completely rational, it has resulted in successful performance. Therefore, it was felt inappropriate to require that structures and elements of structures remain stable for the full lateral forces used in the linear procedures. Instead, the designer must determine if positive direct attachment will be used to resist overturning effects, or if dead loads will be used. If positive direct attachment is to be used, then this attachment is treated just as any other element or component action.

However, if dead loads alone are used to resist overturning, then overturning is treated as a force-controlled behavior and the overturning demands are reduced to an estimate of the real overturning demands which can be transmitted to the element, considering the overall limiting strength of the structure.

There is no rational method available, that has been shown to be consistent with observed behavior, to design or evaluate elements for overturning effects. The method described in the *Guidelines* is rational, but inconsistent with procedures used for new buildings. To improve damage control, the *Guidelines* method is recommended for checking acceptability for Performance Levels higher than Life Safety.

A simplified alternative, described below, for evaluating the adequacy of dead load to provide stability against overturning for Collapse Prevention or Life Safety Performance Levels is to use procedures similar to those used for the design of new buildings:

The load combination represented by

$$Q = 0.9Q_D + \frac{Q_E}{R_{OT}}$$

where Q_D and Q_E have opposite signs, and R_{OT} = 7.5 for Collapse Prevention Performance Level, or 6.0 for Life Safety, is used for evaluating the adequacy of the dead load alone. In the event that the dead load is inadequate, the design of any required hold-downs, piles, or other types of uplift anchors is performed according to the *Guidelines*. Acceptability criteria for components shall be taken from Chapters 5 through 8 with m = 1.

Additional studies are needed on the parameters that control overturning in seismic rehabilitation. These alternative methods are tentative, pending results from this future research.

force at the toe of the element about which the seismic forces tend to cause overturning

C_1, C_2, and C_3 = Coefficients defined in Section 3.3.1.3

J = Coefficient defined in Equation 3-17

The quantity M_{OT}/J need not exceed the overturning moment that can be applied to the element, as limited by the expected strength of the structure responding with an acceptable inelastic mechanism. The element shall be evaluated for the effects of compression on the toe about which it is being overturned. For this purpose, compression at the toe of the element shall be considered a force-controlled action, and shall be evaluated in accordance with the procedures of

Section 3.4.2.1. Refer to Chapter 4 for special considerations related to overturning effects on foundations.

Where dead loads acting on an element are insufficient to provide stability, positive attachment of the element to the structure located above and below the level under consideration shall be provided. These attachments shall be evaluated either as force-controlled or deformation-controlled actions, in accordance with the applicable guidelines provided in Chapters 5 through 8.

2.11.4.2 Nonlinear Procedures

When a nonlinear procedure is followed, the effect of earthquake-induced rocking of elements shall be included in the analytical model as a nonlinear degree of freedom, whenever such rocking can occur. The adequacy of elements above and below the level at which rocking occurs, including the foundations, shall be evaluated for any redistribution of loads that occurs as a result of this rocking in accordance with the procedures of Section 3.4.3.

2.11.5 Continuity

All elements of the structure shall be thoroughly and integrally tied together to form a complete path for the lateral inertial forces generated by the building's response to earthquake demands as follows:

- Every smaller portion of a structure, such as an outstanding wing, shall be tied to the structure as a whole with components capable of resisting horizontal forces equal, at a minimum, to $0.133S_{XS}$ times the weight of the smaller portion of the structure, unless the individual portions of the structure are self-supporting and are separated by a seismic joint.

- Every component shall be connected to the structure to resist a horizontal force in any direction equal, at a minimum, to $0.08S_{XS}$ times the weight of the component. For connections resisting concentrated loads, a minimum force of 1120 pounds shall be used; for distributed load connections, the minimum force shall be 280 pounds per lineal foot.

- Where a sliding support is provided at the end(s) of a component, the bearing length shall be sufficient to accommodate the expected differential displacements of the component relative to its support.

2.11.6 Diaphragms

Diaphragms shall be provided at each level of the structure as necessary to connect building masses to the primary vertical elements of the lateral-force-resisting system. The analytical model used to analyze the building shall account for the behavior of the diaphragms, which shall be evaluated for the forces and displacements indicated by the Analysis Procedure. In addition, the following shall apply:

- **Diaphragm Chords:** Except for diaphragms evaluated as "unchorded" using Chapter 8 of the *Guidelines*, a component shall be provided to develop horizontal shear stresses at each diaphragm edge (either interior or exterior). This component shall consist of either a continuous diaphragm chord, a continuous wall or frame element, or a continuous combination of wall, frame, and chord elements. The forces accumulated in these components and elements due to their action as diaphragm boundaries shall be considered in the evaluation of their adequacy. At re-entrant corners in diaphragms, and at the corners of openings in diaphragms, diaphragm chords shall be extended into the diaphragm a sufficient distance beyond the corner to develop the accumulated diaphragm boundary stresses through the attachment of the extended portion of the chord to the diaphragm.

- **Diaphragm Collectors:** At each vertical element to which a diaphragm is attached, a diaphragm collector shall be provided to transfer to the vertical element those calculated diaphragm forces that cannot be transferred directly by the diaphragm in shear. The diaphragm collector shall be extended into and attached to the diaphragm sufficiently to transfer the required forces.

- **Diaphragm Ties:** Diaphragms shall be provided with continuous tension ties between their chords or boundaries. Ties shall be spaced at a distance not exceeding three times the length of the tie. Ties shall be designed for an axial tensile force equal to $0.4S_{XS}$ times the weight tributary to that portion of the diaphragm located halfway between the tie and each adjacent tie or diaphragm boundary. Where diaphragms of timber, gypsum, or metal deck construction provide lateral support for walls of masonry or concrete construction, ties shall be designed for the wall anchorage forces specified in

Section 2.11.7 for the area of wall tributary to the diaphragm tie.

2.11.7 Walls

Walls shall be anchored to the structure as described in this section, and evaluated for out-of-plane inertial forces as indicated in Chapters 5 through 8.

- Walls shall be positively anchored to all diaphragms that provide lateral support for the wall or are vertically supported by the wall. Walls shall be anchored to diaphragms at horizontal distances not exceeding eight feet, unless it can be demonstrated that the wall has adequate capacity to span longitudinally between the supports for greater distances. Walls shall be anchored to each diaphragm for the larger of $400S_{XS}$ pounds per foot of wall or χS_{XS} times the weight of the wall tributary to the anchor, where χ shall be taken from Table 2-18. The anchorage forces shall be developed into the diaphragm. For flexible diaphragms, the anchorage forces shall be taken as three times those specified above and shall be developed into the diaphragm by continuous diaphragm crossties. For this purpose, diaphragms may be partitioned into a series of subdiaphragms. Each subdiaphragm shall be capable of transmitting the shear forces due to wall anchorage to a continuous diaphragm tie. Subdiaphragms shall have length-to-depth ratios of three or less. Where wall panels are stiffened for out-of-plane behavior by pilasters and similar elements, anchors shall be provided at each such element and the distribution of out-of-plane forces to wall anchors and diaphragm ties shall consider the stiffening effect. Wall anchor connections should be considered force-controlled.

Table 2-18 Coefficient χ for Calculation of Out-of-Plane Wall Forces

Performance Level	χ
Collapse Prevention	0.3
Life Safety	0.4
Immediate Occupancy	0.6

- A wall shall have a strength adequate to span between locations of out-of-plane support when subjected to out-of-plane forces equal to $0.4S_{XS}$ times the unit weight of the wall, over its area.

2.11.8 Nonstructural Components

Nonstructural components, including architectural, mechanical and electrical components, shall be anchored and braced to the structure in accordance with the provisions of Chapter 11. Post-earthquake operability of these components, as required for some Performance Levels, shall also be provided for in accordance with the requirements of Chapter 11 and the project Rehabilitation Objectives.

2.11.9 Structures Sharing Common Elements

Where two or more buildings share common elements, such as party walls or columns, and either the BSO or Enhanced Rehabilitation Objectives are desired, one of the following approaches shall be followed.

- The structures shall be thoroughly tied together so as to behave as an integral unit. Ties between the structures at each level shall be designed for the forces indicated in Section 2.11.5. Analyses of the buildings' response to earthquake demands shall account for the interconnection of the structures and shall evaluate the structures as integral units.

- The buildings shall be completely separated by introducing seismic joints between the structures. Independent lateral-force-resisting systems shall be provided for each structure. Independent vertical support shall be provided on each side of the seismic joint, except that slide bearings to support loads from one structure off the other may be used if adequate bearing length is provided to accommodate the expected independent lateral movement of each structure. It shall be assumed for such purposes that the structures may move out of phase with each other in each direction simultaneously. The original shared element shall be either completely removed or anchored to one of the structures in accordance with the applicable requirements of Section 2.11.5.

2.11.10 Building Separation

2.11.10.1 General

Buildings intended to meet either the BSO or Enhanced Objectives shall be adequately separated from adjacent structures to prevent pounding during response to the design earthquakes, except as indicated in Section 2.11.10.2. Pounding may be presumed not to occur whenever the buildings are separated at any level i by a distance greater than or equal to s_i as given by the equation:

$$s_i = \sqrt{\Delta_{i1}^2 + \Delta_{i2}^2} \tag{2-16}$$

where:

Δ_{i1} = Estimated lateral deflection of building 1 relative to the ground at level i

Δ_{i2} = Estimated lateral deflection of building 2 relative to the ground at level i

The value of s_i calculated by Equation 2-16 need not exceed 0.04 times the height of the buildings above grade at the zone of potential impacts.

2.11.10.2 Special Considerations

Buildings not meeting the separation requirements of Section 2.11.10.1 may be rehabilitated to meet the BSO, subject to the following limitations.

A properly substantiated analysis shall be conducted that accounts for the transfer of momentum and energy between the structures as they impact, and either:

- The diaphragms of the structures shall be located at the same elevations and shall be demonstrated to be capable of transferring the forces resulting from impact; or
- The structures shall be demonstrated to be capable of resisting all required vertical and lateral forces independent of any elements and components that may be severely damaged by impact of the structures.

2.11.11 Vertical Earthquake Effects

The effects of the vertical response of a structure to earthquake ground motion should be considered for any of the following cases:

- Cantilever elements and components of structures
- Pre-stressed elements and components of structures
- Structural components in which demands due to dead and permanent live loads exceed 80% of the nominal capacity of the component

2.12 Quality Assurance

Quality assurance of seismic rehabilitation construction for all buildings and all Rehabilitation Objectives should, as a minimum, conform to the recommendations of this section. These recommendations supplement the recommended testing and inspection requirements contained in the reference standards given in Chapters 5 through 11. The design professional responsible for the seismic rehabilitation of a specific building may find it appropriate to specify more stringent or more detailed requirements. Such additional requirements may be particularly appropriate for those buildings having Enhanced Rehabilitation Objectives.

2.12.1 Construction Quality Assurance Plan

The design professional in responsible charge should prepare a Quality Assurance Plan (QAP) for submittal to the regulatory agency as part of the overall submittal of construction documents. The QAP should specify the seismic-force-resisting elements, components, or systems that are subject to special quality assurance requirements. The QAP should, as a minimum, include the following:

- Required contractor quality control procedures
- Required design professional construction quality assurance services, including but not limited to the following:
  - Review of required contractor submittals
  - Monitoring of required inspection reports and test results
  - Construction consultation as required by the contractor on the intent of the construction documents
  - Procedures for modification of the construction documents to reflect the demands of unforeseen field conditions discovered during construction
  - Construction observation in accordance with Section 2.12.2.1.
- Required special inspection and testing requirements

2.12.2 Construction Quality Assurance Requirements

2.12.2.1 Requirements for the Structural Design Professional

The design professional in responsible charge, or a design professional designated by the design professional in responsible charge, should perform structural observation of the rehabilitation measures shown on the construction documents. Construction observation should include visual observation of the structural system, for general conformance to the conditions assumed during design, and for general conformance to the approved construction documents. Structural observation should be performed at significant construction stages and at completion of the structural/seismic system. Structural construction observation does not include the responsibilities for inspection required by other sections of the *Guidelines*.

Following such structural observations, the structural construction observer should report any observed deficiencies in writing to the owner's representative, the special inspector, the contractor, and the regulatory agency. The structural construction observer should submit to the building official a written statement attesting that the site visits have been made, and identifying any reported deficiencies that, to the best of the structural construction observer's knowledge, have not been resolved or rectified.

2.12.2.2 Special Inspection

The owner should employ a special inspector to observe the construction of the seismic-force-resisting system in accordance with the QAP for the following construction work:

- Items designated in Sections 1.6.2.1 through 1.6.2.9 in the 1994 and 1997 *NEHRP Recommended Provisions* (BSSC, 1995, 1997)
- All other elements and components designated for such special inspection by the design professional
- All other elements and components required by the regulatory agency

2.12.2.3 Testing

The special inspector(s) shall be responsible for verifying that the special test requirements, as described in the QAP, are performed by an approved testing agency for the types of work in the seismic-force-resisting system listed below:

- All work described in Sections 1.6.3.1 through 1.6.3.6 of the 1994 and 1997 *NEHRP Recommended Provisions* (BSSC, 1995, 1997)
- Other types of work designated for such testing by the design professional
- Other types of work required by the regulatory agency

2.12.2.4 Reporting and Compliance Procedures

The special inspector(s) should furnish to the regulatory agency, the design professional in responsible charge, the owner, the persons preparing the QAP, and the contractor copies of progress reports of observations, noting therein any uncorrected deficiencies and corrections of previously reported deficiencies. All observed deficiencies should be brought to the immediate attention of the contractor for correction.

At the completion of construction, the special inspector(s) should submit a final report to the regulatory agency, owner, and design professional in responsible charge indicating the extent to which inspected work was completed in accordance with approved construction documents. Any work not in compliance should be described.

2.12.3 Regulatory Agency Responsibilities

The regulatory agency having jurisdiction over construction of a building that is to be seismically rehabilitated should act to enhance and encourage the protection of the public that is represented by such rehabilitation. These actions should include those described in the following subsections.

2.12.3.1 Construction Document Submittals—Permitting

As part of the permitting process, the regulatory agency should require that construction documents be submitted for a permit to construct the proposed seismic rehabilitation measures. The documents should include a statement of the design basis for the rehabilitation, drawings (or adequately detailed sketches), structural/seismic calculations, and a QAP as recommended by Section 2.12.1. Appropriate structural construction specifications are also recommended, if structural requirements are not adequately defined by notes on drawings.

The regulatory agency should require that it be demonstrated (in the design calculations, by third-party review, or by other means) that the design of the seismic rehabilitation measures has been performed in conformance with local building regulations, the stated design basis, the intent of the *Guidelines*, and/or accepted engineering principles. The regulatory agency should be aware that compliance with the building code provisions for new structures is often not possible nor is it required by the *Guidelines*. It is not intended that the regulatory agency assure compliance of the submittals with the structural requirements for new construction.

The regulatory agency should maintain a permanent public file of the construction documents submitted as part of the permitting process for construction of the seismic rehabilitation measures.

2.12.3.2 Construction Phase Role

The regulatory agency having jurisdiction over the construction of seismic rehabilitation measures should monitor the implementation of the QAP. In particular, the following actions should be taken.

- Files of inspection reports should be maintained for a defined length of time following completion of construction and issuance of a certificate of occupancy. These files should include both reports submitted by special inspectors employed by the owner, as in Section 2.12.2.2, and those submitted by inspectors employed by the regulatory agency.

- Prior to issuance of certificates of occupancy, the regulatory agency should ascertain that either all reported noncompliant aspects of construction have been rectified, or such noncompliant aspects have been accepted by the design professional in responsible charge as acceptable substitutes and consistent with the general intent of the construction documents.

- Files of test reports prepared in accordance with Section 2.12.2.3 should be maintained for a defined length of time following completion of construction and issuance of a certificate of occupancy.

2.13 Alternative Materials and Methods of Construction

When an existing building or rehabilitation scheme contains elements and/or components for which structural modeling parameters and acceptance criteria are not provided in these *Guidelines*, the required parameters and acceptance criteria should be based on the experimentally derived cyclic response characteristics of the assembly, determined in accordance with this section. Independent third-party review of this process, by persons knowledgeable in structural component testing and the derivation of design parameters from such testing, shall be required under this section. The provisions of this section may also be applied to new materials and systems to assess their suitability for seismic rehabilitation.

2.13.1 Experimental Setup

When relevant data on the inelastic force-deformation behavior for a structural subassembly (elements or components) are not available, such data should be obtained based on experiments consisting of physical tests of representative subassemblies. Each subassembly should be an identifiable portion of the structural element or component, the stiffness of which is to be modeled as part of the structural analysis process. The objective of the experiment should be to permit estimation of the lateral-force-displacement relationships (stiffness) for the subassemblies at different loading increments, together with the strength and deformation capacities for the desired performance levels. These properties are to be used in developing an analytical model of the structure’s response to earthquake ground motions, and in judging the acceptability of this predicted behavior. The limiting strength and deformation capacities should be determined from the experimental program from the average values of a minimum of three identical or similar tests performed for a unique design configuration.

The experimental setup should simulate, to the extent practical, the actual construction details, support conditions, and loading conditions expected in the building. Specifically, the effects of axial load, moment, and shear, if expected to be significant in the building, should be properly simulated in the experiments. Full-scale tests are recommended. The loading should consist of fully reversed cyclic loading at increasing displacement levels. The test protocol for number of cycles and displacement levels shall conform to generally accepted procedures. Increments should be continued until the subassembly exhibits complete failure, characterized by a complete (or near-complete) loss of lateral- and gravity-load-resisting ability.

2.13.2 Data Reduction and Reporting

A report should be prepared for each experiment. The report should include the following:

- Description of the subassembly being tested
- Description of the experimental setup, including:
  - Details on fabrication of the subassembly
  - Location and date of experiment
  - Description of instrumentation employed
  - Name of the person in responsible charge of the test
  - Photographs of the specimen, taken prior to testing
- Description of the loading protocol employed, including:
  - Increment of loading (or deformation) applied
  - Rate of loading application
  - Duration of loading at each stage
- Description, including photographic documentation, and limiting deformation value for all important behavior states observed during the test, including the following, as applicable:
  - Elastic range with effective stiffness reported
  - Plastic range
  - Onset of apparent damage
  - Loss of lateral-force-resisting capacity
  - Loss of vertical-load-carrying capacity
  - Force deformation plot for the subassembly (noting the various behavior states)
  - Description of limiting behavior states and failure modes

2.13.3 Design Parameters and Acceptance Criteria

The following procedure should be followed to develop design parameters and acceptance criteria for subassemblies based on experimental data:

1. An idealized lateral-force-deformation pushover curve should be developed from the experimental data for each experiment, and for each direction of loading with unique behavior. The curve should be plotted in a single quadrant (positive force versus positive deformation, or negative force versus negative deformation). The curve should be constructed as follows:

   a. The appropriate quadrant of data from the lateral-force-deformation plot from the experimental report should be taken.

   b. A smooth “backbone” curve should be drawn through the intersection of the first cycle curve for the (i)th deformation step with the second cycle curve of the (i-1)th deformation step, for all i steps, as indicated in Figure 2-6.

   c. The backbone curve so derived shall be approximated by a series of linear segments, drawn to form a multisegmented curve conforming to one of the types indicated in Figure 2-4.

2. The approximate multilinear curves derived for all experiments involving the subassembly should be compared and an average multilinear representation of the subassembly behavior should be derived based on these curves. Each segment of the composite curve should be assigned the average stiffness (either positive or negative) of the similar segments in the approximate multilinear curves for the various experiments. Each segment on the composite curve shall terminate at the average of the deformation levels at which the similar segments of the approximate multilinear curves for the various experiments terminate.

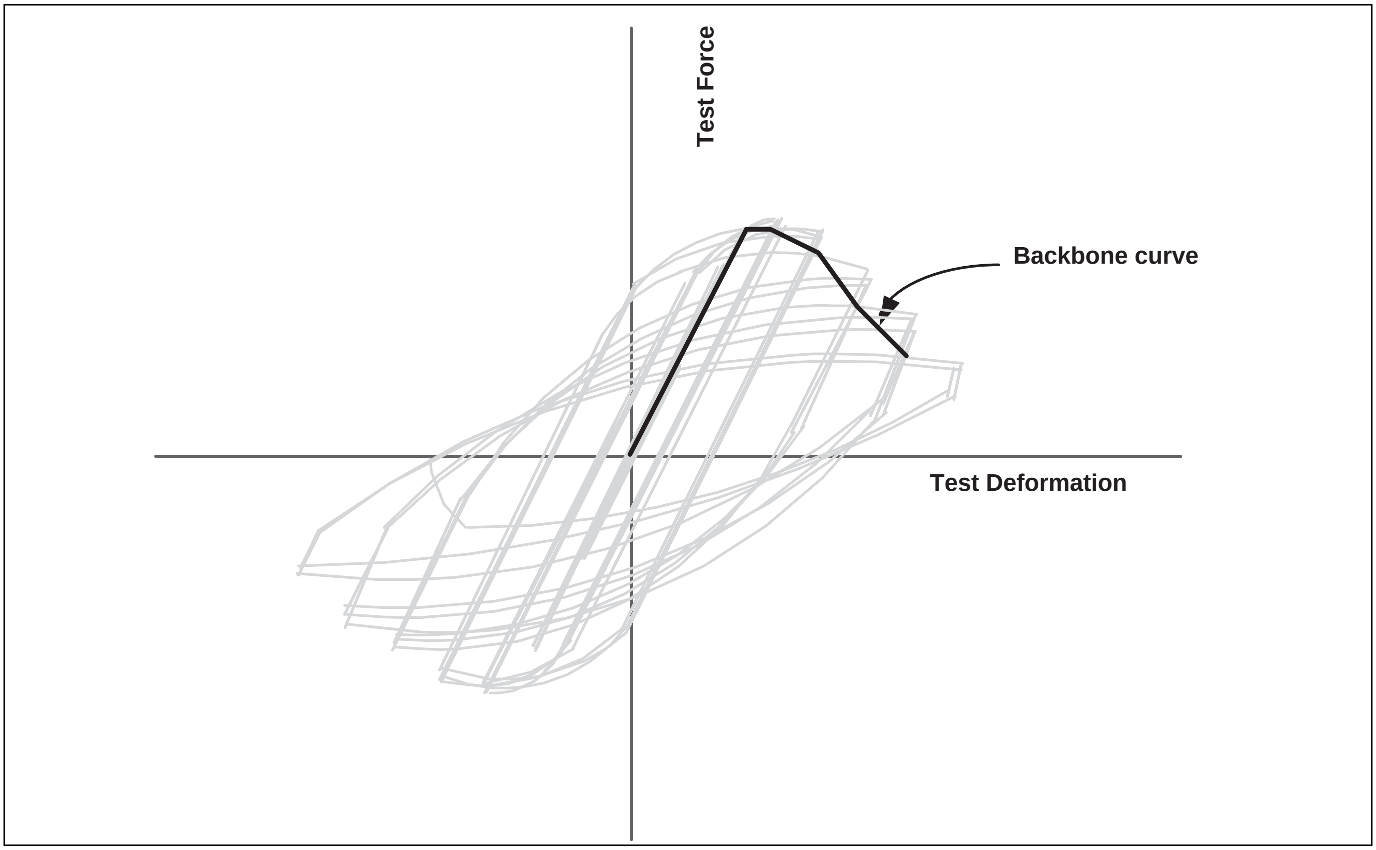

Figure 2-6 Backbone Curve for Experimental Data

3. The stiffness of the subassembly for use in linear procedures should be taken as the slope of the first segment of the composite curve.

4. For the purpose of determining acceptance criteria, assemblies should be classified as being either force-controlled or deformation-controlled. Assemblies should be classified as force-controlled unless any of the following apply.

   – The composite multilinear force-deformation curve for the assembly, determined in accordance with (2), above, conforms to either Type 1 or Type 2, as indicated in Figure 2-4; and the deformation parameter *e*, as indicated in Figure 2-4, is at least twice the deformation parameter *g*, as also indicated in Figure 2-4.

   – The composite multilinear force-deformation curve for the assembly determined in accordance with (2), above, conforms to Type 1, as indicated in Figure 2-4, and the deformation parameter *e* is less than twice the deformation parameter *g*, but the deformation parameter *d* is at least twice the deformation parameter *g*. In this case, acceptance criteria may be determined by redrawing the force-deformation curve as a Type 2 curve, with that portion of the original curve between points 2 and 3 extended back to intersect the first linear segment at point 1' as indicated in Figure 2-7. The parameters a' and Q'_y shall be taken as indicated in Figure 2-7 and shall be used in place of a and Q_y in Figure 2-4.

5. The strength capacity, Q_{CL}, for force-controlled elements evaluated using either the linear or nonlinear procedures shall be taken as follows:

   – For any Performance Level or Range, the lowest strength Q_y determined from the series of representative assembly tests

6. The acceptance criteria for deformation-controlled assemblies used in nonlinear procedures shall be the

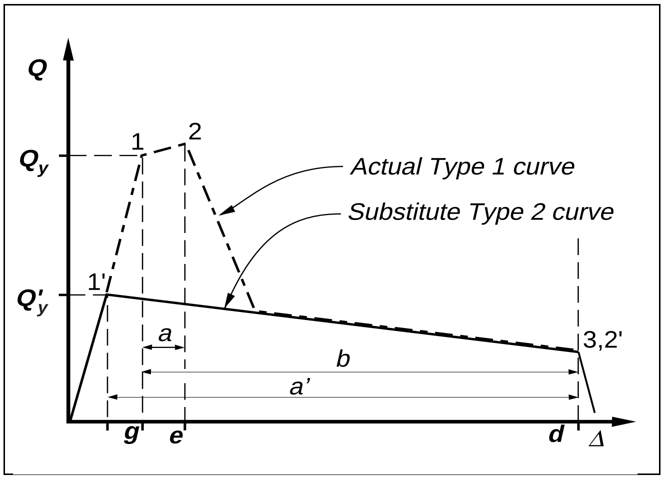

Figure 2-7 Alternative Force Deformation Curve

deformations corresponding with the following points on the curves of Figure 2-4:

a. Primary Elements

- Immediate Occupancy: the deformation at which significant, permanent, visible damage occurred in the experiments
- Life Safety: 0.75 times the deformation at point 2 on the curves
- Collapse Prevention: 0.75 times the deformation at point 3 on the Type 1 curve, but not greater than point 2

b. Secondary Elements

- Immediate Occupancy: the deformation at which significant, permanent, visible damage occurred in the experiments
- Life Safety: 100% of the deformation at point 2 on the Type 1 curve, but not less than 75% of the deformation at point 3
- Collapse Prevention: 100% of the deformation at point 3 on the curve

7. The *m* values used as acceptance criteria for deformation-controlled assemblies in the linear procedures shall be taken as 0.75 times the ratio of the deformation acceptance criteria, given in (6) above, to the deformation at yield, represented by the deformation parameter *g* in the curves shown in Figure 2-4.

2.14 Definitions

Acceptance criteria: Permissible values of such properties as drift, component strength demand, and inelastic deformation used to determine the acceptability of a component's projected behavior at a given Performance Level.

Action: Sometimes called a generalized force, most commonly a single force or moment. However, an action may also be a combination of forces and moments, a distributed loading, or any combination of forces and moments. Actions always produce or cause displacements or deformations; for example, a bending moment action causes flexural deformation in a beam; an axial force action in a column causes axial deformation in the column; and a torsional moment action on a building causes torsional deformations (displacements) in the building.

Assembly: Two or more interconnected components.

BSE-1: Basic Safety Earthquake-1, which is the lesser of the ground shaking at a site for a 10%/50 year earthquake or two-thirds of the Maximum Considered Earthquake (MCE) at the site.

BSE-2: Basic Safety Earthquake-2, which is the ground shaking at a site for an MCE.

BSO: Basic Safety Objective, a Rehabilitation Objective in which the Life Safety Performance Level is reached for the BSE-1 demand and the Collapse Prevention Performance Level is reached for the BSE-2.

Building Performance Level: A limiting damage state, considering structural and nonstructural building components, used in the definition of Rehabilitation Objectives.

Capacity: The permissible strength or deformation for a component action.

Coefficient of variation: For a sample of data, the ratio of the standard deviation for the sample to the mean value for the sample.

Components: The basic structural members that constitute the building, such as beams, columns, slabs, braces, piers, coupling beams, and connections. Components, such as columns and beams, are combined to form elements (e.g., a frame).

Corrective measure: Any modification of a component or element, or the structure as a whole, intended to reduce building vulnerability.

Critical action: That component action that reaches its elastic limit at the lowest level of lateral deflection, or loading, for the structure.

Demand: The amount of force or deformation imposed on an element or component.

Diaphragm: A horizontal (or nearly horizontal) structural element used to distribute inertial lateral forces to vertical elements of the lateral-force-resisting system.

Diaphragm chord: A diaphragm component provided to develop shears at the edge of the diaphragm, resisted either in tension or compression.

Diaphragm collector: A diaphragm component provided to transfer lateral force from the diaphragm to vertical elements of the lateral-force-resisting system or to other portions of the diaphragm.

Element: An assembly of structural components that act together in resisting lateral forces, such as moment-resisting frames, braced frames, shear walls, and diaphragms.

Flexible diaphragm: A diaphragm with stiffness characteristics indicated in Section 3.2.4.

Hazard level: Earthquake shaking demands of specified severity, determined on either a probabilistic or deterministic basis.

Lateral-force-resisting system: Those elements of the structure that provide its basic lateral strength and stiffness, and without which the structure would be laterally unstable.

Maximum Considered Earthquake (MCE): An extreme earthquake hazard level used in the formation of Rehabilitation Objectives. (See BSE-2.)

Mean return period: The average period of time, in years, between the expected occurrences of an earthquake of specified severity.

Nonstructural Performance Level: A limiting damage state for nonstructural building components used to define Rehabilitation Objectives.

Primary component: Those components that are required as part of the building's lateral-force-resisting system (as contrasted to secondary components).

Primary element: An element that is essential to the ability of the structure to resist earthquake-induced deformations.

Rehabilitation Method: A procedural methodology for the reduction of building earthquake vulnerability.

Rehabilitation Objective: A statement of the desired limits of damage or loss for a given seismic demand, usually selected by the owner, engineer, and/or relevant public agencies.

Rehabilitation strategy: A technical approach for developing rehabilitation measures for a building to reduce its earthquake vulnerability.

Secondary component: Those components that are not required for lateral force resistance (contrasted to primary components). They may or may not actually resist some lateral forces.

Secondary element: An element that does not affect the ability of the structure to resist earthquake-induced deformations.

Seismic demand: Seismic hazard level commonly expressed in the form of a ground shaking response spectrum. It may also include an estimate of permanent ground deformation.

Simplified Rehabilitation Method: An approach, applicable to some types of buildings and Rehabilitation Objectives, in which analyses of the entire building's response to earthquake hazards are not required.

Strength: The maximum axial force, shear force, or moment that can be resisted by a component.

Stress resultant: The net axial force, shear, or bending moment imposed on a cross section of a structural component.

Structural Performance Level: A limiting structural damage state, used in the definition of Rehabilitation Objectives.

Structural Performance Range: A range of structural damage states, used in the definition of Rehabilitation Objectives.

Subassembly: A portion of an assembly.

Systematic Rehabilitation Method: An approach to rehabilitation in which complete analysis of the building's response to earthquake shaking is performed.

2.15 Symbols

B_S	Coefficient used to adjust short period spectral response for the effect of viscous damping
B_1	Coefficient used to adjust one-second period spectral response for the effect of viscous damping
C_1	Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response, calculated in accordance with Section 3.3.1.3.
C_2	Modification factor to represent the effect of hysteresis shape on the maximum displacement response, calculated in accordance with Section 3.3.1.3.
C_3	Modification factor to represent increased displacements due to second-order effects, calculated in accordance with Section 3.3.1.3.
DCR	Demand-capacity ratio, computed in accordance with Equation 2-12 or required in Equation 2-13
$\overline{DCR}$	Average demand-capacity ratio for a story, computed in accordance with Equation 2-13
F_a	Factor to adjust spectral acceleration in the short period range for site class
F_v	Factor to adjust spectral acceleration at one second for site class
H	Thickness of a soil layer in feet
J	Coefficient used in linear procedures to estimate the maximum earthquake forces that a component can sustain and correspondingly deliver to other components. The use of J recognizes that the framing system cannot likely deliver the force Q_E because of nonlinear response in the framing system
LDP	Linear Dynamic Procedure—a method of lateral response analysis
LSP	Linear Static Procedure—a method of lateral response analysis
M_{ST}	The stabilizing moment for an element, calculated as the sum of the dead loads acting on the element times the distance between the lines of action of these dead loads and the toe of the element.
M_{OT}	The overting moment on an element, calculated as the sum of the lateral forces applied on the element times the distance between the lines of action of these lateral forces and the toe of the element.
N	Blow count in soil obtained from a standard penetration test (SPT)
$\overline{N}$	Average blow count in soil within the upper 100 feet of soil, calculated in accordance with Equation 2-6
NDP	Nonlinear Dynamic Procedure—a method of lateral response analysis
NSP	Nonlinear Static Procedure—a method of lateral response analysis
P_{E50}	Probability of exceedance in 50 years
PI	Plasticity Index for soil, determined as the difference in water content of soil at the liquid limit and plastic limit
P_i	The total weight of the structure, including dead, permanent live, and 25% of transient live loads acting on the columns and bearing walls within story level i
P_R	Mean return period
Q_{CE}	Expected strength of a component or element at the deformation level under consideration for deformation-controlled actions
Q_{CL}	Lower-bound estimate of the strength of a component or element at the deformation level under consideration for force-controlled actions
Q_D	Calculated stress resultant in a component due to dead load effects

Q_E Calculated earthquake stress resultant in a component

Q_{UD} The dead load force on a component.

Q_y Yield strength of a component

S_S Spectral response acceleration at short periods, obtained from response acceleration maps, *g*

S_{XS} Spectral response acceleration at short periods for any hazard level and any damping, *g*

S_{X1} Spectral response acceleration at a one-second period for any hazard level and any damping, *g*

S_a Spectral acceleration, *g*

S_{aD} Design BSE-1 spectral response acceleration at any period T, *g*

S_{aM} Design BSE-2 spectral response acceleration at any period T, *g*

S_1 Spectral response acceleration at a one-second period, obtained from response acceleration maps, *g*

T Fundamental period of the building in the direction under consideration

T_0 Period at which the constant acceleration and constant velocity regions of the design spectrum intersect

V_i Total calculated lateral shear force in story *i* due to earthquake response, assuming that the structure remains elastic

d_i Depth, in feet, of a layer of soils having similar properties, and located within 100 feet of the surface

h_i Height, in feet, of story *i*; this may be taken as the distance between the centerline of floor framing at each of the levels above and below, the distance between the top of floor slabs at each of the levels above and below, or similar common points of reference

m Modification factor used in the acceptance criteria of deformation-controlled components or elements, indicating the available ductility of a component action

s_i Horizontal distance, in feet, between adjacent buildings at the height above ground at which pounding may occur

s_u Undrained shear strength of soil, pounds/ft^2

$\bar{s}_u$ Average value of the undrained soil shear strength in the upper 100 feet of soil, calculated in accordance with Equation 2-6, pounds/ft^2

v_s Shear wave velocity in soil, in feet/sec

$\bar{v}_s$ Average value of the soil shear wave velocity in the upper 100 feet of soil, calculated in accordance with Equation 2-6, feet/sec

w Water content of soil, calculated as the ratio of the weight of water in a unit volume of soil to the weight of soil in the unit volume, expressed as a percentage

β Modal damping ratio

Δ_{i1} Estimated lateral deflection of building 1 relative to the ground at level *i*

Δ_{i2} Estimated lateral deflection of building 2 relative to the ground at level *i*

δ_i The lateral drift in story *i*, at its center of rigidity

θ_i A parameter indicative of the stability of a structure under gravity loads and earthquake-induced lateral deflection

κ A reliability coefficient used to reduce component strength values for existing components based on the quality of knowledge about the components' properties. (See Section 2.7.2.)

σ Standard deviation of the variation of the material strengths

ϕ A capacity reduction coefficient used to reduce the design strength of new components to account for variations in material strength, cross-section dimension, and construction quality

χ A coefficient used to determine the out-of-plane forces required for anchorage of structural walls to diaphragms

2.16 References

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3. Modeling and Analysis (Systematic Rehabilitation)

3.1 Scope

This chapter presents Analysis Procedures and design requirements for seismic rehabilitation of existing buildings. Section 3.2 presents general requirements for analysis and design that are relevant to all four Analysis Procedures presented in this chapter. The four Analysis Procedures for seismic rehabilitation are presented in Section 3.3, namely: Linear Static Procedure, Linear Dynamic Procedure, Nonlinear Static Procedure, and Nonlinear Dynamic Procedure. Modeling and analysis assumptions, and procedures for determination of design actions and design deformations, are also presented in Section 3.3. Acceptance criteria for elements and components analyzed using any one of the four procedures presented in Section 3.3 are provided in Section 3.4. Section 3.5 provides definitions for key terms used in this chapter, and Section 3.6 defines the symbols used in this chapter. Section 3.7 contains a list of references.

The relationship of the Analysis Procedures described in this chapter with specifications in other chapters in the *Guidelines* is as follows.

- Information on Rehabilitation Objectives to be used for design, including hazard levels (that is, earthquake shaking) and on Performance Levels, is provided in Chapter 2.
- The provisions set forth in this chapter are intended for Systematic Rehabilitation only. Provisions for Simplified Rehabilitation are presented in Chapter 10.
- Guidelines for selecting an appropriate Analysis Procedure are provided in Chapter 2. Chapter 3 describes the loading requirements, mathematical model, and detailed analytical procedures required to estimate seismic force and deformation demands on elements and components of a building. Information on the calculation of appropriate stiffness and strength characteristics for components and elements is provided in Chapters 4 through 9.
- General requirements for analysis and design, including requirements for multidirectional excitation effects, P-Δ effects, torsion, and overturning; basic analysis requirements for the linear and nonlinear procedures; and basic design requirements for diaphragms, walls, continuity of the framing system, building separation, structures sharing common components, and nonstructural components are given in Section 2.11.
- Component strength and deformation demands obtained from analysis using procedures described in this chapter, based on component acceptance criteria outlined in this chapter, are compared with permissible values provided in Chapters 4 through 9 for the desired Performance Level.
- Design methods for walls subjected to out-of-plane seismic forces are addressed in Chapter 2. Analysis and design methods for nonstructural components, and mechanical and electrical equipment, are presented in Chapter 11.
- Specific analysis and design requirements for buildings incorporating seismic isolation and/or supplemental damping hardware are given in Chapter 9.

3.2 General Requirements

Modeling, analysis, and evaluation for Systematic Rehabilitation shall follow the guidelines of this chapter.

3.2.1 Analysis Procedure Selection

Four procedures are presented for seismic analysis of buildings: two linear procedures, and two nonlinear procedures. The two linear procedures are termed the Linear Static Procedure (LSP) and the Linear Dynamic Procedure (LDP). The two nonlinear procedures are termed the Nonlinear Static Procedure (NSP) and Nonlinear Dynamic Procedure (NDP).

Either the linear procedures of Section 3.3.1 and Section 3.3.2, or the nonlinear procedures of Sections 3.3.3 and 3.3.4, may be used to analyze a building, subject to the limitations set forth in Section 2.9.

3.2.2 Mathematical Modeling

3.2.2.1 Basic Assumptions

In general, a building should be modeled, analyzed, and evaluated as a three-dimensional assembly of elements and components. Three-dimensional mathematical models shall be used for analysis and evaluation of buildings with plan irregularity (see Section 3.2.3).

Two-dimensional modeling, analysis, and evaluation of buildings with stiff or rigid diaphragms (see Section 3.2.4) is acceptable if torsional effects are either sufficiently small to be ignored, or indirectly captured (see Section 3.2.2.2).

Vertical lines of seismic framing in buildings with flexible diaphragms (see Section 3.2.4) may be individually modeled, analyzed, and evaluated as two-dimensional assemblies of components and elements, or a three-dimensional model may be used with the diaphragms modeled as flexible elements.

Explicit modeling of a connection is required for nonlinear procedures if the connection is weaker than the connected components, and/or the flexibility of the connection results in a significant increase in the relative deformation between the connected components.

3.2.2.2 Horizontal Torsion

The effects of horizontal torsion must be considered. The total torsional moment at a given floor level shall be set equal to the sum of the following two torsional moments:

- The actual torsion; that is, the moment resulting from the eccentricity between the centers of mass at all floors above and including the given floor, and the center of rigidity of the vertical seismic elements in the story below the given floor, and

- The accidental torsion; that is, an accidental torsional moment produced by horizontal offset in the centers of mass, at all floors above and including the given floor, equal to a minimum of 5% of the horizontal dimension at the given floor level measured perpendicular to the direction of the applied load.

In buildings with rigid diaphragms the effect of actual torsion shall be considered if the maximum lateral displacement from this effect at any point on any floor diaphragm exceeds the average displacement by more than 10%. The effect of accidental torsion shall be considered if the maximum lateral displacement due to this effect at any point on any floor diaphragm exceeds the average displacement by more than 10%. This effect shall be calculated independent of the effect of actual torsion.

If the effects of torsion are required to be investigated, the increased forces and displacements resulting from horizontal torsion shall be evaluated and considered for design. The effects of torsion cannot be used to reduce force and deformation demands on components and elements.

For linear analysis of buildings with rigid diaphragms, when the ratio $\delta_{max} / \delta_{avg}$ due to total torsional moment exceeds 1.2, the effect of accidental torsion shall be amplified by a factor, A_x:

$$A_x = \left(\frac{\delta_{max}}{1.2\delta_{avg}}\right)^2 \qquad (3\text{-}1)$$

where:

δ_{max} = Maximum displacement at any point of the diaphragm at level x

δ_{avg} = Average of displacements at the extreme points of the diaphragm at level x

A_x need not exceed 3.0.

If the ratio η of (1) the maximum displacement at any point on any floor diaphragm (including torsional amplification), to (2) the average displacement, calculated by rational analysis methods, exceeds 1.50, three-dimensional models that account for the spatial distribution of mass and stiffness shall be used for analysis and evaluation. Subject to this limitation, the effects of torsion may be indirectly captured for analysis of two-dimensional models as follows.

- For the LSP (Section 3.3.1) and the LDP (Section 3.3.2), the design forces and displacements shall be increased by multiplying by the maximum value of η calculated for the building.

- For the NSP (Section 3.3.3), the target displacement shall be increased by multiplying by the maximum value of η calculated for the building.
- For the NDP (Section 3.3.4), the amplitude of the ground acceleration record shall be increased by multiplying by the maximum value of η calculated for the building.

3.2.2.3 Primary and Secondary Actions, Components, and Elements

Components, elements, and component actions shall be classified as either primary or secondary. Primary actions, components, and elements are key parts of the seismic framing system required in the design to resist earthquake effects. These shall be evaluated, and rehabilitated as necessary, to sustain earthquake-induced forces and deformations while simultaneously supporting gravity loads. Secondary actions, components, and elements are not designated as part of the lateral-force-resisting system, but nevertheless shall be evaluated, and rehabilitated as necessary, to ensure that such actions, components, and elements can simultaneously sustain earthquake-induced deformations and gravity loads. (See the *Commentary* on this section.)

For linear procedures (Sections 3.3.1 and 3.3.2), only the stiffness of primary components and elements shall be included in the mathematical model. Secondary components and elements shall be checked for the displacements estimated by such analysis. For linear procedures, the total lateral stiffness of the secondary components and elements shall be no greater than 25% of the total stiffness of the primary components and elements, calculated at each level of the building. If this limit is exceeded, some secondary components shall be reclassified as primary components.

For nonlinear procedures (Sections 3.3.3 and 3.3.4), the stiffness and resistance of all primary and secondary components (including strength loss of secondary components) shall be included in the mathematical model. Additionally, if the total stiffness of the nonstructural components—such as precast exterior panels—exceeds 10% of the total lateral stiffness of a story, the nonstructural components shall be included in the mathematical model.

The classification of components and elements shall not result in a change in the classification of a building's configuration (see Section 3.2.3); that is, components and elements shall not be selectively assigned as either primary or secondary to change the configuration of a building from *irregular* to *regular.*

3.2.2.4 Deformation- and Force-Controlled Actions

Actions shall be classified as either deformation-controlled or force-controlled. A deformation-controlled action is one that has an associated deformation that is allowed to exceed the yield value; the maximum associated deformation is limited by the ductility capacity of the component. A force-controlled action is one that has an associated deformation that is not allowed to exceed the yield value. Actions with limited ductility (such as allowing $a < g$ in Figure 2-4) may also be considered force-controlled. Guidance on these classifications may be found in Chapters 5 through 8.

3.2.2.5 Stiffness and Strength Assumptions

Element and component stiffness properties and strength estimates for both linear and nonlinear procedures shall be determined from information given in Chapters 4 through 9, and 11. Guidelines for modeling structural components are given in Chapters 5 through 8. Similar guidelines for modeling foundations and nonstructural components are given in Chapters 4 and 11, respectively.

3.2.2.6 Foundation Modeling

The foundation system may be included in the mathematical model for analysis with stiffness and damping properties as defined in Chapter 4. Otherwise, unless specifically prohibited, the foundation may be assumed to be rigid and not included in the mathematical model.

3.2.3 Configuration

Building irregularities are discussed in Section 2.9. Such classification shall be based on the plan and vertical configuration of the framing system, using a mathematical model that considers both primary and secondary components.

One objective of seismic rehabilitation should be the improvement of the regularity of a building through the judicious placement of new framing elements.

3.2.4 Floor Diaphragms

Floor diaphragms transfer earthquake-induced inertial forces to vertical elements of the seismic framing system. Roof diaphragms are considered to be floor diaphragms. Connections between floor diaphragms and vertical seismic framing elements must have sufficient strength to transfer the maximum calculated diaphragm shear forces to the vertical framing elements. Requirements for design and detailing of diaphragm components are given in Section 2.11.6.

Floor diaphragms shall be classified as either flexible, stiff, or rigid. (See Chapter 10 for classification of diaphragms to be used for determining whether Simplified Rehabilitation Methods are applicable.) Diaphragms shall be considered flexible when the maximum lateral deformation of the diaphragm along its length is more than twice the average interstory drift of the story immediately below the diaphragm. For diaphragms supported by basement walls, the average interstory drift of the story above the diaphragm may be used in lieu of the basement story. Diaphragms shall be considered rigid when the maximum lateral deformation of the diaphragm is less than half the average interstory drift of the associated story. Diaphragms that are neither flexible nor rigid shall be classified as stiff. The interstory drift and diaphragm deformations shall be estimated using the seismic lateral forces (Equation 3-6). The in-plane deflection of the floor diaphragm shall be calculated for an in-plane distribution of lateral force consistent with the distribution of mass, as well as all in-plane lateral forces associated with offsets in the vertical seismic framing at that floor.

Mathematical models of buildings with stiff or flexible diaphragms should be developed considering the effects of diaphragm flexibility. For buildings with flexible diaphragms at each floor level, the vertical lines of seismic framing may be designed independently, with seismic masses assigned on the basis of tributary area.

3.2.5 P-Δ Effects

Two types of P-Δ (second-order) effects are addressed in the *Guidelines*: (1) static P-Δ and (2) dynamic P-Δ.

3.2.5.1 Static P-Δ Effects

For linear procedures, the stability coefficient θ should be evaluated for each story in the building using Equation 2-14. This process is iterative. The story drifts calculated by linear analysis, δ_i in Equation 2-14, shall be increased by $1/(1 - \theta_i)$ for evaluation of the stability coefficient. If the coefficient is less than 0.1 in all stories, static P-Δ effects will be small and may be ignored. If the coefficient exceeds 0.33, the building may be unstable and redesign is necessary (Section 2.11.2). If the coefficient lies between 0.1 and 0.33, the seismic force effects in story *i* shall be increased by the factor $1/(1 - \theta_i)$.

For nonlinear procedures, second-order effects shall be considered directly in the analysis; the geometric stiffness of all elements and components subjected to axial forces shall be included in the mathematical model.

3.2.5.2 Dynamic P-Δ Effects

Dynamic P-Δ effects may increase component actions and deformations, and story drifts. Such effects are indirectly evaluated for the linear procedures and the NSP using the coefficient C_3. Refer to Sections 3.3.1.3A and 3.3.3.3A for additional information.

Second-order effects shall be considered directly for nonlinear procedures; the geometric stiffness of all elements and components subjected to axial forces shall be included in the mathematical model.

3.2.6 Soil-Structure Interaction

Soil-structure interaction (SSI) may modify the seismic demand on a building. Two procedures for computing the effects of SSI are provided below. Other rational methods of modeling SSI may also be used.

For those rare cases (such as for near-field and soft soil sites) in which the increase in fundamental period due to SSI increases spectral accelerations, the effects of SSI on building response must be evaluated; the increase in fundamental period may be calculated using the simplified procedures referred to in Section 3.2.6.1. Otherwise, the effects of SSI may be ignored. In addition, SSI effects need not be considered for any building permitted to be rehabilitated using the Simplified Rehabilitation Method (Table 10-1).

The simplified procedures referred to in Section 3.2.6.1 can be used with the LSP of Section 3.3.1. Consideration of SSI effects with the LDP of Section 3.3.2, the NSP of Section 3.3.3, and the NDP of

Section 3.3.4 shall include explicit modeling of foundation stiffness as in Section 3.2.6.2. Modal damping ratios may be calculated using the method referred to in Section 3.2.6.1.

Soil-structure interaction effects shall not be used to reduce component and element actions by more than 25%.

3.2.6.1 Procedures for Period and Damping

The simplified procedures presented in Chapter 2 of the 1997 *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* (BSSC, 1997) may be used to calculate seismic demands using the effective fundamental period $\tilde{T}$ and effective fundamental damping ratio $\tilde{\beta}$ of the foundation-structure system.

3.2.6.2 Explicit Modeling of SSI

Soil-structure interaction may be modeled explicitly by modeling the stiffness and damping for individual foundation elements. Guidance on the selection of spring characteristics to represent foundation stiffness is presented in Section 4.4.2. Unless otherwise determined, the damping ratio for individual foundation elements shall be set equal to that value of the damping ratio used for the elastic superstructure. For the NSP, the damping ratio of the foundation-structure system $\tilde{\beta}$ shall be used to calculate the spectral demands.

3.2.7 Multidirectional Excitation Effects

Buildings shall be designed for seismic forces in any horizontal direction. For regular buildings, seismic displacements and forces may be assumed to act nonconcurrently in the direction of each principal axis of a building. For buildings with plan irregularity (Section 3.2.3) and buildings in which one or more components form part of two or more intersecting elements, multidirectional excitation effects shall be considered. Multidirectional effects on components shall include both torsional and translational effects.

The requirement that multidirectional (orthogonal) excitation effects be considered may be satisfied by designing elements or components for the forces and deformations associated with 100% of the seismic displacements in one horizontal direction plus the forces associated with 30% of the seismic displacements in the perpendicular horizontal direction. Alternatively, it is acceptable to use SRSS to combine multidirectional effects where appropriate.

The effects of vertical excitation on horizontal cantilevers and prestressed elements shall be considered by static or dynamic response methods. Vertical earthquake shaking may be characterized by a spectrum with ordinates equal to 67% of those of the horizontal spectrum (Section 2.6.1.5) unless alternative vertical response spectra are developed using site-specific analysis.

3.2.8 Component Gravity Loads and Load Combinations

The following component gravity forces, Q_G, shall be considered for combination with seismic loads.

When the effects of gravity and seismic loads are additive,

$$Q_G = 1.1(Q_D + Q_L + Q_S) \tag{3-2}$$

When the effects of gravity counteract seismic loads,

$$Q_G = 0.9Q_D \tag{3-3}$$

where:

Q_D = Dead load effect (action)

Q_L = Effective live load effect (action), equal to 25% of the unreduced design live load but not less than the measured live load

Q_S = Effective snow load effect (action), equal to either 70% of the full design snow load or, where conditions warrant and approved by the regulatory agency, not less than 20% of the full design snow load, except that where the design snow load is 30 pounds per square foot or less, Q_S = 0.0

Evaluation of components for gravity and wind forces, in the absence of earthquake forces, is beyond the scope of this document.

3.2.9 Verification of Design Assumptions

Each component shall be evaluated to determine that assumed locations of inelastic deformations are consistent with strength and equilibrium requirements at all locations along the component length. Further, each component should be evaluated by rational analysis for adequate post-earthquake residual gravity load capacity, considering reduction of stiffness caused by earthquake damage to the structure.

Where moments in horizontally-spanning primary components, due to the gravity load combinations of Equations 3-2 and 3-3, exceed 50% of the expected moment strength at any location, the possibility for inelastic flexural action at locations other than component ends shall be specifically investigated by comparing flexural actions with expected component strengths, and the post-earthquake gravity load capacity should be investigated. Sample checking procedures are presented in the *Commentary*. Formation of flexural plastic hinges away from component ends is not permitted unless it is explicitly accounted for in modeling and analysis.

3.3 Analysis Procedures

3.3.1 Linear Static Procedure (LSP)

3.3.1.1 Basis of the Procedure

Under the Linear Static Procedure (LSP), design seismic forces, their distribution over the height of the building, and the corresponding internal forces and system displacements are determined using a linearly-elastic, static analysis. Restrictions on the applicability of this procedure are given in Section 2.9.

In the LSP, the building is modeled with linearly-elastic stiffness and equivalent viscous damping that approximate values expected for loading to near the yield point. Design earthquake demands for the LSP are represented by static lateral forces whose sum is equal to the pseudo lateral load defined by Equation 3-6. The magnitude of the pseudo lateral load has been selected with the intention that when it is applied to the linearly elastic model of the building it will result in design displacement amplitudes approximating maximum displacements that are expected during the design earthquake. If the building responds essentially elastically to the design earthquake, the calculated internal forces will be reasonable approximations of those expected during the design earthquake. If the building responds inelastically to the design earthquake, as will commonly be the case, the internal forces that would develop in the yielding building will be less than the internal forces calculated on an elastic basis.

Results of the LSP are to be checked using the applicable acceptance criteria of Section 3.4. Calculated internal forces typically will exceed those that the building can develop, because of anticipated inelastic response of components and elements. These obtained design forces are evaluated through the acceptance criteria of Section 3.4.2, which include modification factors and alternative Analysis Procedures to account for anticipated inelastic response demands and capacities.

3.3.1.2 Modeling and Analysis Considerations

Period Determination. The fundamental period of a building, in the direction under consideration, shall be calculated by one of the following three methods. (Method 1 is preferred.)

Method 1. Eigenvalue (dynamic) analysis of the mathematical model of the building. The model for buildings with flexible diaphragms shall consider representation of diaphragm flexibility unless it can be shown that the effects of omission will not be significant.

Method 2. Evaluation of the following equation:

$$T = C_t h_n^{3/4} \tag{3-4}$$

where:

T = Fundamental period (in seconds) in the direction under consideration

C_t = 0.035 for moment-resisting frame systems of steel
= 0.030 for moment-resisting frames of reinforced concrete
= 0.030 for eccentrically-braced steel frames
= 0.020 for all other framing systems
= 0.060 for wood buildings (types 1 and 2 in Table 10-2)

h_n = Height (in feet) above the base to the roof level

Method 2 is not applicable to unreinforced masonry buildings with flexible diaphragms.

Method 3. The fundamental period of a one-story building with a single span flexible diaphragm may be calculated as:

$$T = (0.1\Delta_w + 0.078\Delta_d)^{0.5} \qquad (3\text{-}5)$$

where Δ_w and Δ_d are in-plane wall and diaphragm displacements in inches, due to a lateral load, in the direction under consideration, equal to the weight tributary to the diaphragm (see *Commentary*, Figure C3-2). For multiple-span diaphragms, a lateral load equal to the gravity weight tributary to the diaphragm span under consideration shall be applied to each diaphragm span to calculate a separate period for each diaphragm span. The period so calculated that maximizes the pseudo lateral load (see Equation 3-6) shall be used for design of all walls and diaphragm spans in the building.

3.3.1.3 Determination of Actions and Deformations

A. Pseudo Lateral Load

The pseudo lateral load in a given horizontal direction of a building is determined using Equation 3-6. This load, increased as necessary to account for the effects of torsion (see Section 3.2.2.2), shall be used for the design of the vertical seismic framing system.

$$V = C_1 C_2 C_3 S_a W \qquad (3\text{-}6)$$

where:

V = Pseudo lateral load

This force, when distributed over the height of the linearly-elastic analysis model of the structure, is intended to produce calculated lateral displacements approximately equal to those that are expected in the real structure during the design event. If it is expected that the actual structure will yield during the design event, the force given by Equation 3-6 may be significantly larger than the actual strength of the structure to resist this force. The acceptance criteria in Section 3.4.2 are developed to take this aspect into account.

C_1 = Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response. C_1 may be calculated using the procedure indicated in Section 3.3.3.3. with the elastic base shear capacity substituted for V_y. Alternatively, C_1 may be calculated as follows:

$C_1 = 1.5$ for $T < 0.10$ second

$C_1 = 1.0$ for $T \geq T_0$ second

Linear interpolation shall be used to calculate C_1 for intermediate values of T.

T = Fundamental period of the building in the direction under consideration. If soil-structure interaction is considered, the effective fundamental period $\tilde{T}$ shall be substituted for T.

T_0 = Characteristic period of the response spectrum, defined as the period associated with the transition from the constant acceleration segment of the spectrum to the constant velocity segment of the spectrum. (See Sections 2.6.1.5 and 2.6.2.1.)

C_2 = Modification factor to represent the effect of stiffness degradation and strength deterioration on maximum displacement response. Values of C_2 for different framing systems and Performance Levels are listed in Table 3-1. Linear interpolation shall be used to estimate values for C_2 for intermediate values of T.

C_3 = Modification factor to represent increased displacements due to dynamic P-Δ effects. This effect is in addition to the consideration of static P-D effects as defined in Section 3.2.5.1. For values of the stability coefficient θ (see Equation 2-14) less than 0.1, C_3 may be set equal to 1.0. For values of θ greater than 0.1, C_3 shall be calculated as $1 + 5(\theta - 0.1)/T$. The maximum value of θ for all stories in the building shall be used to calculate C_3.

S_a = Response spectrum acceleration, at the fundamental period and damping ratio of the building in the direction under consideration. The value of S_a shall be obtained from the procedure in Section 2.6.1.5.

W = Total dead load and anticipated live load as indicated below:

- In storage and warehouse occupancies, a minimum of 25% of the floor live load
- The actual partition weight or minimum weight of 10 psf of floor area, whichever is greater
- The applicable snow load—see the *NEHRP Recommended Provisions* (BSSC, 1995)
- The total weight of permanent equipment and furnishings

B. Vertical Distribution of Seismic Forces

The lateral load F_x applied at any floor level *x* shall be determined from the following equations:

$$F_x = C_{vx}V \tag{3-7}$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \tag{3-8}$$

where:

k = 1.0 for $T \le 0.5$ second
= 2.0 for $T \ge 2.5$ seconds
Linear interpolation shall be used to estimate values of k for intermediate values of T.

C_{vx} = Vertical distribution factor

V = Pseudo lateral load from Equation 3-6

w_i = Portion of the total building weight *W* located on or assigned to floor level *i*

w_x = Portion of the total building weight *W* located on or assigned to floor level *x*

h_i = Height (in ft) from the base to floor level *i*

h_x = Height (in ft) from the base to floor level *x*

C. Horizontal Distribution of Seismic Forces

The seismic forces at each floor level of the building shall be distributed according to the distribution of mass at that floor level.

D. Floor Diaphragms

Floor diaphragms shall be designed to resist the effects of (1) the inertia forces developed at the level under consideration (equal to F_{px} in Equation 3-9), and (2) the horizontal forces resulting from offsets in, or changes in stiffness of, the vertical seismic framing elements above and below the diaphragm. Forces resulting from offsets in, or changes in stiffness of, the vertical seismic framing elements shall be taken to be equal to the elastic forces (Equation 3-6) without reduction, unless smaller forces can be justified by rational analysis.

$$F_{px} = \frac{1}{C_1 C_2 C_3} \sum_{i=x}^{n} F_i \frac{w_x}{\sum_{i=x}^{n} w_i} \tag{3-9}$$

where:

F_{px} = Total diaphragm force at level *x*

F_i = Lateral load applied at floor level *i* given by Equation 3-7

w_i = Portion of the total building weight *W* located on or assigned to floor level *i*

w_x = Portion of the total building weight *W* located on or assigned to floor level *x*

Coefficients C_1, C_2, and C_3 are described above in Section 3.3.1.3A.

The lateral seismic load on each flexible diaphragm shall be distributed along the span of that diaphragm, considering its displaced shape.

E. Determination of Deformations

Structural deformations and story drifts shall be calculated using lateral loads in accordance with Equations 3-6, 3-7, and 3-9 and stiffnesses obtained from Chapters 5, 6, 7, and 8.

Table 3-1 Values for Modification Factor C_2

Performance Level	$T = 0.1$ **second**		$T \geq T_0$ **second**	
	Framing Type 1[1]	**Framing Type 2**[2]	**Framing Type 1**[1]	**Framing Type 2**[2]
Immediate Occupancy	1.0	1.0	1.0	1.0
Life Safety	1.3	1.0	1.1	1.0
Collapse Prevention	1.5	1.0	1.2	1.0

1. Structures in which more than 30% of the story shear at any level is resisted by components or elements whose strength and stiffness may deteriorate during the design earthquake. Such elements and components include: ordinary moment-resisting frames, concentrically-braced frames, frames with partially-restrained connections, tension-only braced frames, unreinforced masonry walls, shear-critical walls and piers, or any combination of the above.
2. All frames not assigned to Framing Type 1.

3.3.2 Linear Dynamic Procedure (LDP)

3.3.2.1 Basis of the Procedure

Under the Linear Dynamic Procedure (LDP), design seismic forces, their distribution over the height of the building, and the corresponding internal forces and system displacements are determined using a linearly-elastic, dynamic analysis. Restrictions on the applicability of this procedure are given in Section 2.9.

The basis, modeling approaches, and acceptance criteria of the LDP are similar to those for the LSP. The main exception is that the response calculations are carried out using either modal spectral analysis or Time-History Analysis. Modal spectral analysis is carried out using linearly-elastic response spectra that are not modified to account for anticipated nonlinear response. As with the LSP, it is expected that the LDP will produce displacements that are approximately correct, but will produce internal forces that exceed those that would be obtained in a yielding building.

Results of the LDP are to be checked using the applicable acceptance criteria of Section 3.4. Calculated displacements are compared directly with allowable values. Calculated internal forces typically will exceed those that the building can sustain because of anticipated inelastic response of components and elements. These obtained design forces are evaluated through the acceptance criteria of Section 3.4.2, which include modification factors and alternative analysis procedures to account for anticipated inelastic response demands and capacities.

3.3.2.2 Modeling and Analysis Considerations

A. General

The LDP shall conform to the criteria of this section. The analysis shall be based on appropriate characterization of the ground motion (Section 2.6.1). The modeling and analysis considerations set forth in Section 3.3.1.2 shall apply to the LDP but alternative considerations are presented below.

The LDP includes two analysis methods, namely, the Response Spectrum and Time-History Analysis Methods. The Response Spectrum Method uses peak modal responses calculated from dynamic analysis of a mathematical model. Only those modes contributing significantly to the response need to be considered. Modal responses are combined using rational methods to estimate total building response quantities. The Time-History Method (also termed Response-History Analysis) involves a time-step-by-time-step evaluation of building response, using discretized recorded or synthetic earthquake records as base motion input. Requirements for the two analysis methods are outlined in C and D below.

B. Ground Motion Characterization

The horizontal ground motion shall be characterized for design by the requirements of Section 2.6 and shall be one of the following:

- A response spectrum (Section 2.6.1.5)
- A site-specific response spectrum (Section 2.6.2.1)
- Ground acceleration time histories (Section 2.6.2.2)

C. Response Spectrum Method

The requirement that all significant modes be included in the response analysis may be satisfied by including sufficient modes to capture at least 90% of the participating mass of the building in each of the building's principal horizontal directions. Modal damping ratios shall reflect the damping inherent in the building at deformation levels less than the yield deformation.

The peak member forces, displacements, story forces, story shears, and base reactions for each mode of response shall be combined by recognized methods to estimate total response. Modal combination by either the SRSS (square root sum of squares) rule or the CQC (complete quadratic combination) rule is acceptable.

Multidirectional excitation effects shall be accounted for by the requirements of Section 3.2.7.

D. Time-History Method

The requirements for the mathematical model for Time-History Analysis are identical to those developed for Response Spectrum Analysis. The damping matrix associated with the mathematical model shall reflect the damping inherent in the building at deformation levels less than the yield deformation.

Time-History Analysis shall be performed using time histories prepared according to the requirements of Section 2.6.2.2.

Response parameters shall be calculated for each Time-History Analysis. If three Time-History Analyses are performed, the maximum response of the parameter of interest shall be used for design. If seven or more pairs of horizontal ground motion records are used for Time-History Analysis, the average response of the parameter of interest may be used for design.

Multidirectional excitation effects shall be accounted for in accordance with the requirements of Section 3.2.7. These requirements may be satisfied by analysis of a three-dimensional mathematical model using simultaneously imposed pairs of earthquake ground motion records along each of the horizontal axes of the building.

3.3.2.3 Determination of Actions and Deformations

A. Modification of Demands

All actions and deformations calculated using either of the LDP analysis methods—Response Spectrum or Time-History Analysis—shall be multiplied by the product of the modification factors C_1, C_2, and C_3 defined in Section 3.3.1.3, and further increased as necessary to account for the effects of torsion (see Section 3.2.2.2). However, floor diaphragm actions need not be increased by the product of the modification factors.

B. Floor Diaphragms

Floor diaphragms shall be designed to resist simultaneously (1) the seismic forces calculated by the LDP, and (2) the horizontal forces resulting from offsets in, or changes in stiffness of, the vertical seismic framing elements above and below the diaphragm. The seismic forces calculated by the LDP shall be taken as not less than 85% of the forces calculated using Equation 3-9. Forces resulting from offsets in, or changes in stiffness of, the vertical seismic framing elements shall be taken to be equal to the elastic forces without reduction, unless smaller forces can be justified by rational analysis.

3.3.3 Nonlinear Static Procedure (NSP)

3.3.3.1 Basis of the Procedure

Under the Nonlinear Static Procedure (NSP), a model directly incorporating inelastic material response is displaced to a target displacement, and resulting internal deformations and forces are determined. The nonlinear load-deformation characteristics of individual components and elements of the building are modeled directly. The mathematical model of the building is subjected to monotonically increasing lateral forces or displacements until either a target displacement is exceeded or the building collapses. The target displacement is intended to represent the maximum displacement likely to be experienced during the design earthquake. The target displacement may be calculated by any procedure that accounts for the effects of nonlinear response on displacement amplitude; one rational procedure is presented in Section 3.3.3.3. Because the mathematical model accounts directly for effects of material inelastic response, the calculated internal forces will be reasonable approximations of those expected during the design earthquake.

Results of the NSP are to be checked using the applicable acceptance criteria of Section 3.4.3. Calculated displacements and internal forces are compared directly with allowable values.

3.3.3.2 Modeling and Analysis Considerations

A. General

In the context of these *Guidelines*, the NSP involves the monotonic application of lateral forces or displacements to a nonlinear mathematical model of a building until the displacement of the control node in the mathematical model exceeds a target displacement. For buildings that are not symmetric about a plane perpendicular to the applied lateral loads, the lateral loads must be applied in both the positive and negative directions, and the maximum forces and deformations used for design.

The relation between base shear force and lateral displacement of the control node shall be established for control node displacements ranging between zero and 150% of the target displacement, δ_t, given by Equation 3-11. Acceptance criteria shall be based on those forces and deformations (in components and elements) corresponding to a minimum horizontal displacement of the control node equal to δ_t.

Gravity loads shall be applied to appropriate elements and components of the mathematical model during the NSP. The loads and load combination presented in Equation 3-2 (and Equation 3-3 as appropriate) shall be used to represent such gravity loads.

The analysis model shall be discretized in sufficient detail to represent adequately the load-deformation response of each component along its length. Particular attention should be paid to identifying locations of inelastic action along the length of a component, as well as at its ends.

B. Control Node

The NSP requires definition of the control node in a building. These *Guidelines* consider the control node to be the center of mass at the roof of a building; the top of a penthouse should not be considered as the roof. The displacement of the control node is compared with the target displacement—a displacement that characterizes the effects of earthquake shaking.

C. Lateral Load Patterns

Lateral loads shall be applied to the building in profiles that approximately bound the likely distribution of inertia forces in an earthquake. For three-dimensional analysis, the horizontal distribution should simulate the distribution of inertia forces in the plane of each floor diaphragm. For both two- and three-dimensional analysis, at least two vertical distributions of lateral load shall be considered. The first pattern, often termed the uniform pattern, shall be based on lateral forces that are proportional to the total mass at each floor level. The second pattern, termed the modal pattern in these *Guidelines*, should be selected from one of the following two options:

- a lateral load pattern represented by values of C_{vx} given in Equation 3-8, which may be used if more than 75% of the total mass participates in the fundamental mode in the direction under consideration; or

- a lateral load pattern proportional to the story inertia forces consistent with the story shear distribution calculated by combination of modal responses using (1) Response Spectrum Analysis of the building including a sufficient number of modes to capture 90% of the total mass, and (2) the appropriate ground motion spectrum.

D. Period Determination

The effective fundamental period T_e in the direction under consideration shall be calculated using the force-displacement relationship of the NSP. The nonlinear relation between base shear and displacement of the target node shall be replaced with a bilinear relation to estimate the effective lateral stiffness, K_e, and the yield strength, V_y, of the building. The effective lateral stiffness shall be taken as the secant stiffness calculated at a base shear force equal to 60% of the yield strength. The effective fundamental period T_e shall be calculated as:

$$T_e = T_i\sqrt{\frac{K_i}{K_e}} \tag{3-10}$$

where:

T_i = Elastic fundamental period (in seconds) in the direction under consideration calculated by elastic dynamic analysis

K_i = Elastic lateral stiffness of the building in the direction under consideration

K_e = Effective lateral stiffness of the building in the direction under consideration

See Figure 3-1 for further information.

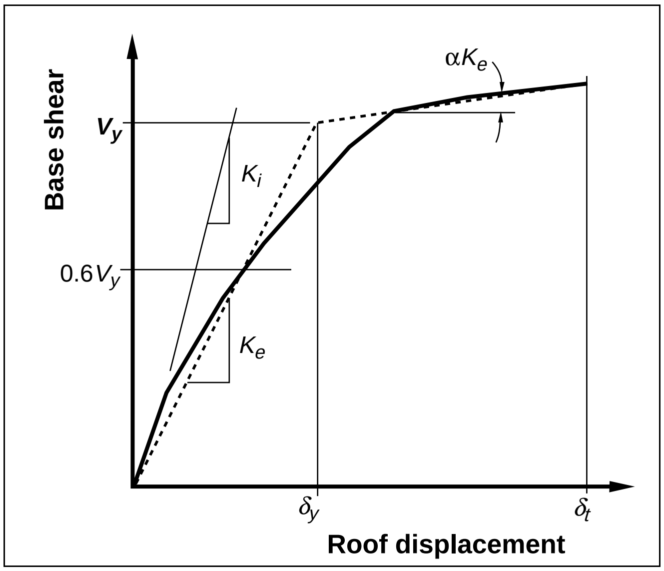


Figure 3-1 Calculation of Effective Stiffness, K_e

E. Analysis of Three-Dimensional Models

Static lateral forces shall be imposed on the three-dimensional mathematical model corresponding to the mass distribution at each floor level. The effects of accidental torsion shall be considered (Section 3.2.2.2).

Independent analysis along each principal axis of the three-dimensional mathematical model is permitted unless multidirectional evaluation is required (Section 3.2.7).

F. Analysis of Two-Dimensional Models

Mathematical models describing the framing along each axis (axis 1 and axis 2) of the building shall be developed for two-dimensional analysis. The effects of horizontal torsion shall be considered (Section 3.2.2.2).

If multidirectional excitation effects are to be considered, component deformation demands and actions shall be computed for the following cases: 100% of the target displacement along axis 1 and 30% of the target displacement along axis 2; and 30% of the target displacement along axis 1 and 100% of the target displacement along axis 2.

3.3.3.3 Determination of Actions and Deformations

A. Target Displacement

The target displacement δ_t for a building with rigid diaphragms (Section 3.2.4) at each floor level shall be estimated using an established procedure that accounts for the likely nonlinear response of the building.

Actions and deformations corresponding to the control node displacement equaling or exceeding the target displacement shall be used for component checking in Section 3.4.

One procedure for evaluating the target displacement is given by the following equation:

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} g \tag{3-11}$$

where:

T_e = Effective fundamental period of the building in the direction under consideration, sec

C_0 = Modification factor to relate spectral displacement and likely building roof displacement

Estimates for C_0 can be calculated using one of the following:

- the first modal participation factor at the level of the control node
- the modal participation factor at the level of the control node calculated using a shape vector corresponding to the deflected shape of the building at the target displacement
- the appropriate value from Table 3-2

C_1 = Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response

= 1.0 for $T_e \geq T_0$

= $[1.0 + (R-1)T_0/T_e]/R$ for $T_e < T_0$

Values for C_1 need not exceed those values given in Section 3.3.1.3.
In no case may C_1 be taken as less than 1.0.

T_0 = Characteristic period of the response spectrum, defined as the period associated with the transition from the constant acceleration segment of the spectrum to the constant velocity segment of the spectrum. (See Sections 2.6.1.5 and 2.6.2.1.)

R = Ratio of elastic strength demand to calculated yield strength coefficient. See below for additional information.

C_2 = Modification factor to represent the effect of hysteresis shape on the maximum displacement response. Values for C_2 are established in Section 3.3.1.3.

C_3 = Modification factor to represent increased displacements due to dynamic P-Δ effects. For buildings with positive post-yield stiffness, C_3 shall be set equal to 1.0. For buildings with negative post-yield stiffness, values of C_3 shall be calculated using Equation 3-13. Values for C_3 need not exceed the values set forth in Section 3.3.1.3.

S_a = Response spectrum acceleration, at the effective fundamental period and damping ratio of the building in the direction under consideration, g. The value of S_a is calculated in Sections 2.6.1.5 and 2.6.2.1.

The strength ratio R shall be calculated as:

$$R = \frac{S_a}{V_y/W} \cdot \frac{1}{C_0} \tag{3-12}$$

Table 3-2 Values for Modification Factor C_0

Number of Stories	Modification Factor[1]
1	1.0
2	1.2
3	1.3
5	1.4
10+	1.5

1. Linear interpolation should be used to calculate intermediate values.

where S_a and C_0 are as defined above, and:

V_y = Yield strength calculated using results of NSP, where the nonlinear force-displacement (i.e., base shear force versus control node displacement) curve of the building is characterized by a bilinear relation (Figure 3-1)

W = Total dead load and anticipated live load, as calculated in Section 3.3.1.3

Coefficient C_3 shall be calculated as follows if the relation between base shear force and control node displacement exhibits negative post-yield stiffness.

$$C_3 = 1.0 + \frac{|\alpha|(R-1)^{3/2}}{T_e} \tag{3-13}$$

where R and T_e are as defined above, and:

α = Ratio of post-yield stiffness to effective elastic stiffness, where the nonlinear force-displacement relation is characterized by a bilinear relation (Figure 3-1)

For a building with flexible diaphragms (Section 3.2.4) at each floor level, a target displacement shall be estimated for each line of vertical seismic framing. The target displacements shall be estimated using an established procedure that accounts for the likely nonlinear response of the seismic framing. One procedure for evaluating the target displacement for an individual line of vertical seismic framing is given by Equation 3-11. The fundamental period of each vertical line of seismic framing, for calculation of the target displacement, shall follow the general procedures described for the NSP; masses shall be assigned to each level of the mathematical model on the basis of tributary area.

For a building with neither rigid nor flexible diaphragms at each floor level, the target displacement shall be calculated using rational procedures. One acceptable procedure for including the effects of diaphragm flexibility is to multiply the displacement calculated using Equation 3-11 by the ratio of the maximum displacement at any point on the roof and the displacement of the center of mass of the roof, both calculated by modal analysis of a three-dimensional model of the building using the design response spectrum. The target displacement so calculated shall be no less than that displacement given by Equation 3-11, assuming rigid diaphragms at each floor level. No vertical line of seismic framing shall be evaluated for displacements smaller than the target displacement. The target displacement should be modified according to Section 3.2.2.2 to account for system torsion.

B. Floor Diaphragms

Floor diaphragms may be designed to resist simultaneously both the seismic forces determined using either Section 3.3.1.3D or Section 3.3.2.3B, and the horizontal forces resulting from offsets in, or changes in stiffness of, the vertical seismic framing elements above and below the diaphragm.

3.3.4 Nonlinear Dynamic Procedure (NDP)

3.3.4.1 Basis of the Procedure

Under the Nonlinear Dynamic Procedure (NDP), design seismic forces, their distribution over the height of the building, and the corresponding internal forces and system displacements are determined using an inelastic response history dynamic analysis.

The basis, modeling approaches, and acceptance criteria of the NDP are similar to those for the NSP. The main exception is that the response calculations are carried out using Time-History Analysis. With the NDP, the design displacements are not established using a target displacement, but instead are determined directly through dynamic analysis using ground motion histories. Calculated response can be highly sensitive to characteristics of individual ground motions; therefore, it is recommended to carry out the analysis with more than one ground motion record. Because the numerical model accounts directly for effects of material inelastic response, the calculated internal forces will be reasonable approximations of those expected during the design earthquake.

Results of the NDP are to be checked using the applicable acceptance criteria of Section 3.4. Calculated displacements and internal forces are compared directly with allowable values.

3.3.4.2 Modeling and Analysis Assumptions

A. General

The NDP shall conform to the criteria of this section. The analysis shall be based on characterization of the seismic hazard in the form of ground motion records (Section 2.6.2). The modeling and analysis considerations set forth in Section 3.3.3.2 shall apply to the NDP unless the alternative considerations presented below are applied.

The NDP requires Time-History Analysis of a nonlinear mathematical model of the building, involving a time-step-by-time-step evaluation of building response, using discretized recorded or synthetic earthquake records as base motion input.

B. Ground Motion Characterization

The earthquake shaking shall be characterized by ground motion time histories meeting the requirements of Section 2.6.2.

C. Time-History Method

Time-History Analysis shall be performed using horizontal ground motion time histories prepared according to the requirements of Section 2.6.2.2.

Multidirectional excitation effects shall be accounted for by meeting the requirements of Section 3.2.7. The requirements of Section 3.2.7 may be satisfied by analysis of a three-dimensional mathematical model using simultaneously imposed pairs of earthquake ground motion records along each of the horizontal axes of the building.

3.3.4.3 Determination of Actions and Deformations

A. Modification of Demands

The effects of torsion shall be considered according to Section 3.2.2.2.

B. Floor Diaphragms

Floor diaphragms shall be designed to resist simultaneously both the seismic forces calculated by dynamic analysis and the horizontal forces resulting from offsets in, or changes in stiffness of, the vertical seismic framing elements above and below the diaphragm.

3.4 Acceptance Criteria

3.4.1 General Requirements

Components and elements analyzed using the linear procedures of Sections 3.3.1 and 3.3.2 shall satisfy the requirements of this section and Section 3.4.2. Components and elements analyzed using the nonlinear procedures of Sections 3.3.3 and 3.3.4 shall satisfy the requirements of this section and Section 3.4.3.

For the purpose of evaluating acceptability, actions shall be categorized as being either deformation-controlled or force-controlled, as defined in Section 3.2.2.4.

Foundations shall satisfy the criteria set forth in Chapter 4.

3.4.2 Linear Procedures

3.4.2.1 Design Actions

A. Deformation-Controlled Actions

Design actions Q_{UD} shall be calculated according to Equation 3-14.

$$Q_{UD} = Q_G \pm Q_E \qquad (3\text{-}14)$$

where:

Q_E = Action due to design earthquake loads calculated using forces and analysis models described in either Section 3.3.1 or Section 3.3.2

Q_G = Action due to design gravity loads as defined in Section 3.2.8

Q_{UD} = Design action due to gravity loads and earthquake loads

B. Force-Controlled Actions

The value of a force-controlled design action Q_{UF} need not exceed the maximum action that can be developed in a component considering the nonlinear behavior of the building. It is recommended that this value be based on limit analysis. In lieu of more rational analysis, design actions may be calculated according to Equation 3-15 or Equation 3-16.

$$Q_{UF} = Q_G \pm \frac{Q_E}{C_1 C_2 C_3 J} \qquad (3\text{-}15)$$

$$Q_{UF} = Q_G \pm \frac{Q_E}{C_1 C_2 C_3} \qquad (3\text{-}16)$$

where:

Q_{UF} = Design actions due to gravity loads and earthquake loads

J = Force-delivery reduction factor given by Equation 3-17

Equation 3-16 can be used in all cases. Equation 3-15 can only be used if the forces contributing to Q_{UF} are delivered by yielding components of the seismic framing system.

The coefficient J shall be established using Equation 3-17.

$$J = 1.0 + S_{XS}\text{, not to exceed 2} \qquad (3\text{-}17)$$

where:

S_{XS} = Spectral acceleration, calculated in Section 2.6.1.4

Alternatively, J may be taken as equal to the smallest DCR of the components in the load path delivering force to the component in question.

3.4.2.2 Acceptance Criteria for Linear Procedures

A. Deformation-Controlled Actions

Deformation-controlled actions in primary and secondary components and elements shall satisfy Equation 3-18.

$$m\kappa Q_{CE} \geq Q_{UD} \tag{3-18}$$

where:

m = Component or element demand modifier to account for expected ductility of the deformation associated with this action at selected Performance Level (see Chapters 4 through 8)

Q_{CE} = Expected strength of the component or element at the deformation level under consideration for deformation-controlled actions

κ = Knowledge factor (Section 2.7.2)

For Q_{CE}, the expected strength shall be determined considering all coexisting actions acting on the component under the design loading condition. Procedures to determine the expected strength are given in Chapters 4 through 8.

B. Force-Controlled Actions

Force-controlled actions in primary and secondary components and elements shall satisfy Equation 3-19.

$$\kappa Q_{CL} \geq Q_{UF} \tag{3-19}$$

where:

Q_{CL} = Lower-bound strength of a component or element at the deformation level under consideration for force-controlled actions

For Q_{CL}, the lower-bound strength shall be determined considering all coexisting actions acting on the component under the design loading condition. Procedures to determine the lower-bound strength are specified in Chapters 5 through 8.

C. Verification of Design Assumptions

Each component shall be evaluated to determine that assumed locations of inelastic deformations are consistent with strength and equilibrium requirements at all locations along the component length.

Where moments due to gravity loads in horizontally-spanning primary components exceed 75% of the expected moment strength at any location, the possibility for inelastic flexural action at locations other than member ends shall be specifically investigated by comparing flexural actions with expected member strengths. Formation of flexural plastic hinges away from member ends shall not be permitted where design is based on the LSP or the LDP.

3.4.3 Nonlinear Procedures

3.4.3.1 Design Actions and Deformations

Design actions (forces and moments) and deformations shall be the maximum values determined from the NSP or the NDP, whichever is applied.

3.4.3.2 Acceptance Criteria for Nonlinear Procedures

A. Deformation-Controlled Actions

Primary and secondary components shall have expected deformation capacities not less than the maximum deformations. Expected deformation capacities shall be determined considering all coexisting forces and deformations. Procedures for determining expected deformation capacities are specified in Chapters 5 through 8.

B. Force-Controlled Actions

Primary and secondary components shall have lower-bound strengths Q_{CL} not less than the maximum design actions. Lower-bound strength shall be determined considering all coexisting forces and deformations. Procedures for determining lower-bound strengths are specified in Chapters 5 through 8.

3.5 Definitions

This section provides definitions for all key terms used in this chapter and not previously defined.

Action: Sometimes called a generalized force, most commonly a single force or moment. However, an action may also be a combination of forces and moments, a distributed loading, or any combination of forces and moments. Actions always produce or cause displacements or deformations. For example, a bending moment action causes flexural deformation in a beam; an axial force action in a column causes axial deformation in the column; and a torsional moment action on a building causes torsional deformations (displacements) in the building.

Base: The level at which earthquake effects are considered to be imparted to the building.

Components: The basic structural members that constitute the building, such as beams, columns, slabs, braces, piers, coupling beams, and connections. Components, such as columns and beams, are combined to form elements (e.g., a frame).

Control node: The node in the mathematical model of a building used to characterize mass and earthquake displacement.

Deformation: Relative displacement or rotation of the ends of a component or element.

Displacement: The total movement, typically horizontal, of a component or element or node.

Flexible diaphragm: A diaphragm that meets requirements of Section 3.2.4.

Framing type: Type of seismic resisting system.

Element: An assembly of structural components that act together in resisting lateral forces, such as moment-resisting frames, braced frames, shear walls, and diaphragms.

Fundamental period: The first mode period of the building in the direction under consideration.

Inter-story drift: The relative horizontal displacement of two adjacent floors in a building. Inter-story drift can also be expressed as a percentage of the story height separating the two adjacent floors.

Primary component: Those components that are required as part of the building's lateral-force-resisting system (as contrasted to secondary components).

Rigid diaphragm: A diaphragm that meets requirements of Section 3.2.4

Secondary component: Those components that are not required for lateral force resistance (contrasted to primary components). They may or may not actually resist some lateral forces.

Stiff diaphragm: A diaphragm that meets requirements of Section 3.2.4.

Target displacement: An estimate of the likely building roof displacement in the design earthquake.

3.6 Symbols

This section provides symbols for all key variables used in this chapter and not defined previously.

C_0	Modification factor to relate spectral displacement and likely building roof displacement
C_1	Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response
C_2	Modification factor to represent the effect of hysteresis shape on the maximum displacement response
C_3	Modification factor to represent increased displacements due to second-order effects
C_t	Numerical values following Equation 3-4
C_{vx}	Vertical distribution factor for the pseudo lateral load
F_d	Total lateral load applied to a single bay of a diaphragm
F_i and F_x	Lateral load applied at floor levels *i* and *x*, respectively
F_{px}	Diaphragm lateral force at floor level *x*
J	A coefficient used in linear procedures to estimate the actual forces delivered to force-controlled components by other (yielding) components.
K_e	Effective stiffness of the building in the direction under consideration, for use with the NSP
K_i	Elastic stiffness of the building in the direction under consideration, for use with the NSP
L_d	Single-bay diaphragm span
Q_{CE}	Expected strength of a component or element at the deformation level under consideration in a deformation-controlled action

Q_{CL}	Lower-bound estimate of the strength of a component or element at the deformation level under consideration for force-controlled actions
Q_D	Dead load force (action)
Q_E	Earthquake force (action) calculated using procedures of Section 3.3.1 or 3.3.2
Q_G	Gravity load force (action)
Q_L	Effective live load force (action)
Q_S	Effective snow load force (action)
Q_{UD}	Deformation-controlled design action
Q_{UF}	Force-controlled design action
R	Ratio of the elastic strength demand to the yield strength coefficient
S_a	Response spectrum acceleration at the fundamental period and damping ratio of the building, g
S_{XS}	Spectral response acceleration at short periods for any hazard level and damping, g
T	Fundamental period of the building in the direction under consideration
T_e	Effective fundamental period of the building in the direction under consideration, for use with the NSP
T_i	Elastic fundamental period of the building in the direction under consideration, for use with the NSP
T_0	Period at which the constant acceleration and constant velocity regions of the design spectrum intersect
V	Pseudo lateral load
V_y	Yield strength of the building in the direction under consideration, for use with the NSP
W	Total dead load and anticipated live load
W_i and W_x	Weight of floors i and x, respectively
f_d	Lateral load per foot of diaphragm span
g	Acceleration of gravity (386.1 in./sec^2, or 9,807 mm/sec^2 for SI units)
h_i and h_x	Height from the base of a building to floor levels i and x, respectively
h_n	Height to roof level, ft
k	Exponent used for determining the vertical distribution of lateral forces
m	A modification factor used in the acceptance criteria of deformation-controlled components or elements, indicating the available ductility of a component action
w_i and w_x	Portion of the total building weight corresponding to floor levels i and x, respectively
x	Distance from the diaphragm center line
Δ_d	Diaphragm deformation
Δ_w	Average in-plane wall displacement
α	Ratio of post-yield stiffness to effective stiffness
δ_t	Target roof displacement
δ_y	Yield displacement of building (Figure 3-1)
η	Displacement multiplier, greater than 1.0, to account for the effects of torsion
θ	Stability coefficient (Equation 2-14)—a parameter indicative of the stability of a structure under gravity loads and earthquake-induced deflection
κ	Reliability coefficient used to reduce component strength values for existing components based on the quality of knowledge about the components' properties. (See Section 2.7.2.)

3.7 References

BSSC, 1995, *NEHRP Recommended Provisions for Seismic Regulations for New Buildings, 1994 Edition, Part 1: Provisions and Part 2: Commentary*, prepared by the Building Seismic Safety Council, for the Federal Emergency Management Agency (Report Nos. FEMA 222A and 223A), Washington, D.C.

BSSC, 1997, *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, 1997 Edition, Part 1: Provisions and Part 2:*

Commentary, prepared by the Building Seismic Safety Council for the Federal Emergency Management Agency (Report Nos. FEMA 302 and 303), Washington, D.C.

4. Foundations and Geotechnical Hazards (Systematic Rehabilitation)

4.1 Scope

This chapter provides geotechnical engineering guidance regarding building foundations and seismic-geologic site hazards. Acceptability of the behavior of the foundation system and foundation soils for a given Performance Level cannot be determined apart from the context of the behavior of the superstructure.

Geotechnical requirements for buildings that are suitable for Simplified Rehabilitation are included in Chapter 10.

Structural engineering issues of foundation systems are discussed in the chapters on Steel (Chapter 5), Concrete (Chapter 6), Masonry (Chapter 7), and Wood (Chapter 8).

This chapter describes rehabilitation measures for foundations and geotechnical site hazards. Section 4.2 provides guidelines for establishing site soil characteristics and identifying geotechnical site hazards, including fault rupture, liquefaction, differential compaction, landslide and rock fall, and flooding. Techniques for mitigating these geotechnical site hazards are described in Section 4.3. Section 4.4 presents criteria for establishing soil strength capacity, stiffness, and soil-structure interaction (SSI) parameters for making foundation design evaluations. Retaining walls are discussed in Section 4.5. Section 4.6 contains guidelines for improving or strengthening foundations.

4.2 Site Characterization

The geotechnical requirements for buildings suitable for Simplified Rehabilitation are described in Chapter 10. For all other buildings, specific geotechnical site characterization consistent with the selected method of Systematic Rehabilitation is required. Site characterization consists of the compilation of information on site subsurface soil conditions, configuration and loading of existing building foundations, and seismic-geologic site hazards.

In the case of historic buildings, the guidance of the State Historic Preservation Officer should be obtained if historic or archeological resources are present at the site.

4.2.1 Foundation Soil Information

Specific information describing the foundation conditions of the building to be rehabilitated is required. Useful information also can be gained from knowledge of the foundations of adjacent or nearby buildings. Foundation information may include subsurface soil and ground water data, configuration of the foundation system, design foundation loads, and load-deformation characteristics of the foundation soils.

4.2.1.1 Site Foundation Conditions

Subsurface soil conditions must be defined in sufficient detail to assess the ultimate capacity of the foundation and to determine if the site is susceptible to seismic-geologic hazards.

Information regarding the structural foundation type, dimensions, and material are required irrespective of the subsurface soil conditions. This information includes:

- Foundation type—spread footings, mat foundation, piles, drilled shafts.
- Foundation dimensions—plan dimensions and locations. For piles, tip elevations, vertical variations (tapered sections of piles or belled caissons).
- Material composition/construction. For piles, type (concrete/steel/wood), and installation method (cast-in-place, open/closed-end driving).

Subsurface conditions shall be determined for the selected Performance Level as follows.

A. Collapse Prevention and Life Safety Performance Levels

Determine type, composition, consistency, relative density, and layering of soils to a depth at which the stress imposed by the building is approximately 10% of the building weight divided by the total foundation area.

Determine the location of the water table and its seasonal fluctuations beneath the building.

B. Enhanced Rehabilitation Objectives and/or Deep Foundations

For each soil type, determine soil unit weight γ, soil shear strength c, soil friction angle ϕ, soil compressibility characteristics, soil shear modulus G, and Poisson's ratio ν.

4.2.1.2 Nearby Foundation Conditions

Specific foundation information developed for an adjacent or nearby building may be useful if subsurface soils and ground water conditions in the site region are known to be uniform. However, less confidence will result if subsurface data are developed from anywhere but the site being rehabilitated. Adjacent sites where construction has been done recently may provide a guide for evaluation of subsurface conditions at the site being considered.

4.2.1.3 Design Foundation Loads

Information on the design foundation loads is required, as well as actual dead loads and realistic estimates of live loads.

4.2.1.4 Load-Deformation Characteristics Under Seismic Loading

Traditional geotechnical engineering treats load-deformation characteristics for long-term dead loads plus frequently applied live loads only. In most cases, long-term settlement governs foundation design. Short-term (earthquake) load-deformation characteristics have not traditionally been used for design; consequently, such relationships are not generally found in the soils and foundation reports for existing buildings. Load-deformation relationships are discussed in detail in Section 4.4.

4.2.2 Seismic Site Hazards

In addition to ground shaking, seismic hazards include surface fault rupture, liquefaction, differential compaction, landsliding, and flooding. The potential for ground displacement hazards at a site should be evaluated. The evaluation should include an assessment of the hazards in terms of ground movement. If consequences are unacceptable for the desired Performance Level, then the hazards should be mitigated as described in Section 4.3.

4.2.2.1 Fault Rupture

Geologic site conditions must be defined in sufficient detail to assess the potential for the trace of an active fault to be present in the building foundation soils. If the trace of a fault is known or suspected to be present, the following information may be required:

- The degree of activity—that is, the age of most recent movement (e.g., historic, Holocene, late Quaternary)—must be determined.
- The fault type must be identified, whether strike-slip, normal-slip, reverse-slip, or thrust fault.
- The sense of slip with respect to building geometry must be determined, particularly for normal-slip and reverse-slip faults.
- Magnitudes of vertical and/or horizontal displacements with recurrence intervals consistent with Rehabilitation Objectives must be determined.
- The width of the fault-rupture zone (concentrated in a narrow zone or distributed) must be identified.

4.2.2.2 Liquefaction

Subsurface soil and ground water conditions must be defined in sufficient detail to assess the potential for liquefiable materials to be present in the building foundation soils. If liquefiable soils are suspected to be present, the following information must be developed.

- **Soil type:** Liquefiable soils typically are granular (sand, silty sand, nonplastic silt).
- **Soil density:** Liquefiable soils are loose to medium dense.
- **Depth to water table:** Liquefiable soils must be saturated, but seasonal fluctuations of the water table must be estimated.
- **Ground surface slope and proximity of free-face conditions:** Lateral-spread landslides can occur on gently sloping sites, particularly if a free-face condition—such as a canal or stream channel—is present nearby.
- **Lateral and vertical differential displacement:** Amount and direction at the building foundation must be calculated.

The hazard of liquefaction should be evaluated initially to ascertain whether the site is clearly free of a hazardous condition or whether a more detailed evaluation is required. It can be assumed generally that a significant hazard due to liquefaction does not exist at a site if the site soils or similar soils in the site vicinity have not experienced historical liquefaction and if any of the following criteria are met:

- The geologic materials underlying the site are either bedrock or have a very low liquefaction susceptibility, according to the relative susceptibility ratings based upon general depositional environment and geologic age of the deposit, as shown in Table 4-1.

Table 4-1 Estimated Susceptibility to Liquefaction of Surficial Deposits During Strong Ground Shaking (after Youd and Perkins, 1978)

Type of Deposit	General Distribution of Cohesionless Sediments in Deposits	Likelihood that Cohesionless Sediments, When Saturated, Would be Susceptible to Liquefaction (by Age of Deposit)			
		Modern < 500 yr.	Holocene < 11,000 yr.	Pleistocene < 2 million yr.	Pre-Pleistocene > 2 million yr.
(a) Continental Deposits					
River channel	Locally variable	Very high	High	Low	Very low
Flood plain	Locally variable	High	Moderate	Low	Very low
Alluvial fan, plain	Widespread	Moderate	Low	Low	Very low
Marine terrace	Widespread	—	Low	Very low	Very low
Delta, fan delta	Widespread	High	Moderate	Low	Very low
Lacustrine, playa	Variable	High	Moderate	Low	Very low
Colluvium	Variable	High	Moderate	Low	Very low
Talus	Widespread	Low	Low	Very low	Very low
Dune	Widespread	High	Moderate	Low	Very low
Loess	Variable	High	High	High	Unknown
Glacial till	Variable	Low	Low	Very low	Very low
Tuff	Rare	Low	Low	Very low	Very low
Tephra	Widespread	High	Low	?	?
Residual soils	Rare	Low	High	Very low	Very low
Sebka	Locally variable	High	Moderate	Low	Very low
(b) Coastal Zone Deposits					
Delta	Widespread	Very high	High	Low	Very low
Esturine	Locally variable	High	Moderate	Low	Very low
Beach, high energy	Widespread	Moderate	Low	Very low	Very low
Beach, low energy	Widespread	High	Moderate	Low	Very low
Lagoon	Locally variable	High	Moderate	Low	Very low
Foreshore	Locally variable	High	Moderate	Low	Very low
(c) Fill Materials					
Uncompacted fill	Variable	Very high	—	—	—
Compacted fill	Variable	Low	—	—	—

- The soils underlying the site are stiff clays or clayey silts, unless the soils are highly sensitive, based on local experience; or, the soils are cohesionless (i.e., sand, silts, or gravels) with a minimum normalized Standard Penetration Test (SPT) resistance, $(N_1)_{60}$, value of 30 blows/foot for depths below the groundwater table, or with clay content greater than 20%. The parameter $(N_1)_{60}$ is defined as the SPT blow count normalized to an effective overburden pressure of 2 ksf. Clay has soil particles with nominal diameters $\leq$ 0.005 mm.

- The groundwater table is at least 35 feet below the deepest foundation depth, or 50 feet below the ground surface, whichever is shallower, including considerations for seasonal and historic ground-water level rises, and any slopes or free-face conditions in the site vicinity do not extend below the ground-water elevation at the site.

If, by applying the above criteria, a possible liquefaction hazard at the site cannot be eliminated, then a more detailed evaluation is required. Guidance for detailed evaluations is presented in the *Commentary.*

4.2.2.3 Differential Compaction

Subsurface soil conditions must be defined in sufficient detail to assess the potential for differential compaction to occur in the building foundation soils.

Differential compaction or densification of soils may accompany strong ground shaking. The resulting differential settlements can be damaging to structures. Types of soil that are susceptible to liquefaction (that is, relatively loose natural soils, or uncompacted or poorly compacted fill soils) are also susceptible to compaction. Compaction can occur in soils above and below the groundwater table.

It can generally be assumed that a significant hazard due to differential compaction does not exist if the soil conditions meet both of the following criteria:

- The geologic materials underlying foundations and below the groundwater table do not pose a significant liquefaction hazard, based on the criteria in Section 4.2.2.2.

- The geologic materials underlying foundations and above the groundwater table are either Pleistocene in geologic age (older than 11,000 years), stiff clays or clayey silts, or cohesionless sands, silts, and gravels with a minimum $(N_1)_{60}$ of 20 blows/0.3 m (20 blows/foot).

If a possible differential compaction hazard at the site cannot be eliminated by applying the above criteria, then a more detailed evaluation is required. Guidance for a detailed evaluation is presented in the *Commentary.*

4.2.2.4 Landsliding

Subsurface soil conditions must be defined in sufficient detail to assess the potential for a landslide to cause differential movement of the building foundation soils. Hillside stability shall be evaluated at sites with:

- Existing slopes exceeding approximately 18 degrees (three horizontal to one vertical)

- Prior histories of instability (rotational or translational slides, or rock fall)

Pseudo-static analyses shall be used to determine site stability, provided the soils are not liquefiable or otherwise expected to lose shear strength during deformation. Pseudo-static analyses shall use a seismic coefficient equal to one-half the peak ground acceleration (calculated as $S_{XS}/2.5$) at the site associated with the desired Rehabilitation Objective. Sites with a static factor of safety equal to or greater than 1.0 shall be judged to have adequate stability, and require no further stability analysis.

Sites with a static factor of safety of less than 1.0 will require a sliding-block displacement analysis (Newmark, 1965). The displacement analysis shall determine the magnitude of potential ground movement for use by the structural engineer in determining its effect upon the performance of the structure and the structure's ability to meet the desired Performance Level. Where the structural performance cannot accommodate the computed ground displacements, appropriate mitigation schemes shall be employed as described in Section 4.3.4.

In addition to potential effects of landslides on foundation soils, the possible effects of rock fall or slide debris from adjacent slopes should be considered.

4.2.2.5 Flooding or Inundation

For Performance Levels exceeding Life Safety, site conditions should be defined in sufficient detail to assess the potential for earthquake-induced flooding or inundation to prevent the rehabilitated building from meeting the desired Performance Level. Sources of earthquake-induced flooding or inundation include:

- Dams located upstream damaged by earthquake shaking or fault rupture
- Pipelines, aqueducts, and water-storage tanks located upstream damaged by fault rupture, earthquake-induced landslides, or strong shaking
- Low-lying coastal areas within tsunami zones or areas adjacent to bays or lakes that may be subject to seiche waves
- Low-lying areas with shallow ground water where regional subsidence could cause surface ponding of water, resulting in inundation of the site

Potential damage to buildings from flooding or inundation must be evaluated on a site-specific basis. Consideration must be given to potential scour of building foundation soils from swiftly flowing water.

4.3 Mitigation of Seismic Site Hazards

Opportunities exist to improve seismic performance under the influence of some site hazards at reasonable cost; however, some site hazards may be so severe that they are economically impractical to include in risk-reduction measures. The discussions presented below are based on the concept that the extent of site hazards is discovered after the decision for seismic rehabilitation of a building has been made; however, the decision to rehabilitate a building and the selection of a Rehabilitation Objective may have been made with full knowledge that significant site hazards exist and must be mitigated as part of the rehabilitation.

4.3.1 Fault Rupture

Large movements caused by fault rupture generally cannot be mitigated economically. If the structural consequences of the estimated horizontal and vertical displacements are unacceptable for any Performance Level, either the structure, its foundation, or both, might be stiffened or strengthened to reach acceptable performance. Measures are highly dependent on specific structural characteristics and inadequacies. Grade beams and reinforced slabs are effective in increasing resistance to horizontal displacement. Horizontal forces are sometimes limited by sliding friction capacity of spread footings or mats. Vertical displacements are similar in nature to those caused by long-term differential settlement. Mitigative techniques include modifications to the structure or its foundation to distribute the effects of differential vertical movement over a greater horizontal distance to reduce angular distortion.

4.3.2 Liquefaction

The effectiveness of mitigating liquefaction hazards must be evaluated by the structural engineer in the context of the global building system performance. If it has been determined that liquefaction is likely to occur and the consequences in terms of estimated horizontal and vertical displacements are unacceptable for the desired Performance Level, then three general types of mitigating measures can be considered alone or in combination.

Modify the structure: The structure can be strengthened to improve resistance against the predicted liquefaction-induced ground deformation. This solution may be feasible for small ground deformations.

Modify the foundation: The foundation system can be modified to reduce or eliminate the potential for large foundation displacements; for example, by underpinning existing shallow foundations to achieve bearing on deeper, nonliquefiable strata. Alternatively (or in concert with the use of deep foundations), a shallow foundation system can be made more rigid (for example, by a system of grade beams between isolated footings) in order to reduce the differential ground movements transmitted to the structure.

Modify the soil conditions: A number of types of ground improvement can be considered to reduce or eliminate the potential for liquefaction and its effects. Techniques that generally are potentially applicable to existing buildings include soil grouting, either throughout the entire liquefiable strata beneath a building, or locally beneath foundation elements (e.g., grouted soil columns); installation of drains (e.g., stone columns); and installation of permanent dewatering systems. Other types of ground improvement that are widely used for new construction are less applicable to existing buildings because of the effects of the procedures on the building. Thus, removal and replacement of liquefiable soil or in-place densification of liquefiable soil by various techniques are not applicable beneath an existing building.

If potential for significant liquefaction-induced lateral spreading movements exists at a site, then the remediation of the liquefaction hazard may be more difficult. This is because the potential for lateral spreading movements beneath a building may depend on the behavior of the soil mass at distances well beyond the building as well as immediately beneath it. Thus, measures to prevent lateral spreading may, in some cases, require stabilizing large soil volumes and/or constructing buttressing structures that can reduce the potential for, or the amount of, lateral movements.

4.3.3 Differential Compaction

The effectiveness of mitigating differential compaction hazards must be evaluated by the structural engineer in the context of the global building system performance. For cases of predicted significant differential settlements of a building foundation, mitigation options are similar to those described above to mitigate liquefaction hazards. There are three options: designing for the ground movements, strengthening the foundation system, and improving the soil conditions.

4.3.4 Landslide

The effectiveness of mitigating landslide hazards must be evaluated by the structural engineer in the context of the global building system performance. A number of schemes are available for reducing potential impacts for earthquake-induced landslides, including:

- Regrading
- Drainage
- Buttressing
- Structural Improvements
  - Gravity walls
  - Tieback/soil nail walls
  - Mechanically stabilized earth walls
  - Barriers for debris torrents or rock fall
  - Building strengthening to resist deformation
    - Grade beams
    - Shear walls
- Soil Modification/Replacement
  - Grouting
  - Densification

The effectiveness of any of these schemes must be considered based upon the amount of ground movement that the building can tolerate and still meet the desired Performance Level.

4.3.5 Flooding or Inundation

The effectiveness of mitigating flooding or inundation hazards must be evaluated by the structural engineer in the context of the global building system performance. Potential damage caused by earthquake-induced flooding or inundation may be mitigated by a number of schemes, as follows:

- Improvement of nearby dam, pipeline, or aqueduct facilities independent of the rehabilitated building
- Diversion of anticipated peak flood flows
- Installation of pavement around the building to minimize scour
- Construction of sea wall or breakwater for tsunami or seiche protection

4.4 Foundation Strength and Stiffness

It is assumed in this section that the foundation soils are not susceptible to significant strength loss due to earthquake loading. With this assumption, the following paragraphs provide an overview of the requirements and procedures for evaluating the ability of foundations to withstand the imposed seismic loads without excessive deformations. If soils are susceptible to significant strength loss, due to either the direct effects of the earthquake shaking on the soil or the foundation loading on the soil induced by the earthquake, then either improvement of the soil foundation condition should be considered or special analyses should be carried out to demonstrate that the effects of soil strength loss do not result in excessive structural deformations.

Consideration of foundation behavior is only one part of seismic rehabilitation of buildings. Selection of the desired Rehabilitation Objective probably will be done without regard to specific details of the building, including the foundation. The structural engineer will choose the appropriate type of analysis procedures for the selected Performance Level (e.g., Systematic Rehabilitation, with Linear Static or Dynamic Procedures, or Nonlinear Static or Dynamic Procedures). As stated previously, foundation requirements for buildings that qualify for Simplified Rehabilitation are included in Chapter 10.

4.4.1 Ultimate Bearing Capacities and Load Capacities

The ultimate load capacity of foundation components may be determined by one of the three methods specified below. The choice of method depends on the completeness of available information on foundation properties (see Section 4.2.1.1) and the requirements of the selected Performance Level.

4.4.1.1 Presumptive Ultimate Capacities

Presumptive capacities are to be used when the amount of information on foundation soil properties is limited and relatively simple analysis procedures are used. Presumptive ultimate load parameters for spread footings and mats are presented in Table 4-2.

4.4.1.2 Prescriptive Ultimate Capacities

Prescriptive capacities may be used when either construction documents for the existing building or previous geotechnical reports provide information on foundation soils design parameters.

The ultimate prescriptive bearing pressure for a spread footing may be assumed to be twice the allowable dead plus live load bearing pressure specified for design.

$$q_c = 2q_{allow.D+L} \tag{4-1}$$

Table 4-2 Presumptive Ultimate Foundation Pressures

Class of Materials[2]	Vertical Foundation Pressure[3] Lbs./Sq. Ft. (q_c)	Lateral Bearing Pressure Lbs./Sq. Ft./Ft. of Depth Below Natural Grade[4]	Lateral Sliding[1] Coefficient[5]	Lateral Sliding[1] Resistance[6] Lbs./Sq. Ft.
Massive Crystalline Bedrock	8000	2400	0.80	—
Sedimentary and Foliated Rock	4000	800	0.70	—
Sandy Gravel and/or Gravel (GW and GP)	4000	400	0.70	—
Sand, Silty Sand, Clayey Sand, Silty Gravel, and Clayey Gravel (SW, SP, SM, SC, GM, and GC)	3000	300	0.50	—
Clay, Sandy Clay, Silty Clay, and Clayey Silt (CL, ML, MH, and CH)	2000[7]	200	—	260

1. Lateral bearing and lateral sliding resistance may be combined.
2. For soil classifications OL, OH, and PT (i.e., organic clays and peat), a foundation investigation shall be required.
3. All values of ultimate foundation pressure are for footings having a minimum width of 12 inches and a minimum depth of 12 inches into natural grade. Except where Footnote 7 below applies, increase of 20% allowed for each additional foot of width or depth to a maximum value of three times the designated value.
4. May be increased by the amount of the designated value for each additional foot of depth to a maximum of 15 times the designated value.
5. Coefficient applied to the dead load.
6. Lateral sliding resistance value to be multiplied by the contact area. In no case shall the lateral sliding resistance exceed one-half the dead load.
7. No increase for width is allowed.

For deep foundations, the ultimate prescriptive vertical capacity of individual piles or piers may be assumed to be 50% greater than the allowable dead plus live loads specified for design.

$$Q_c = 1.5Q_{allow.D+L} \tag{4-2}$$

As an alternative, the prescriptive ultimate capacity of any footing component may be assumed to be 50% greater than the total working load acting on the component, based on analyses using the original design requirements.

$$Q_c = 1.5Q_{max.} \tag{4-3}$$

where $Q_{max.} = Q_D + Q_L + Q_S$

4.4.1.3 Site-Specific Capacities

A detailed analysis may be conducted by a qualified geotechnical engineer to determine ultimate foundation capacities based on the specific characteristics of the building site.

4.4.2 Load-Deformation Characteristics for Foundations

Load-deformation characteristics are required where the effects of foundations are to be taken into account in Linear Static or Dynamic Procedures (LSP or LDP), Nonlinear Static (pushover) Procedures (NSP), or Nonlinear Dynamic (time-history) Procedures (NDP). Foundation load-deformation parameters characterized by both stiffness and capacity can have a significant effect on both structural response and load distribution among structural elements.

Foundation systems for buildings can in some cases be complex, but for the purpose of simplicity, three foundation types are considered in these *Guidelines*:

- shallow bearing foundations
- pile foundations
- drilled shafts

While it is recognized that the load-deformation behavior of foundations is nonlinear, because of the difficulties in determining soil properties and static foundation loads for existing buildings, together with the likely variability of soils supporting foundations, an equivalent elasto-plastic representation of load-deformation behavior is recommended. In addition, to allow for such variability or uncertainty, an upper and lower bound approach to defining stiffness and capacity is recommended (as shown in Figure 4-1a) to permit evaluation of structural response sensitivity. The selection of uncertainty represented by the upper and lower bounds should be determined jointly by the geotechnical and structural engineers.

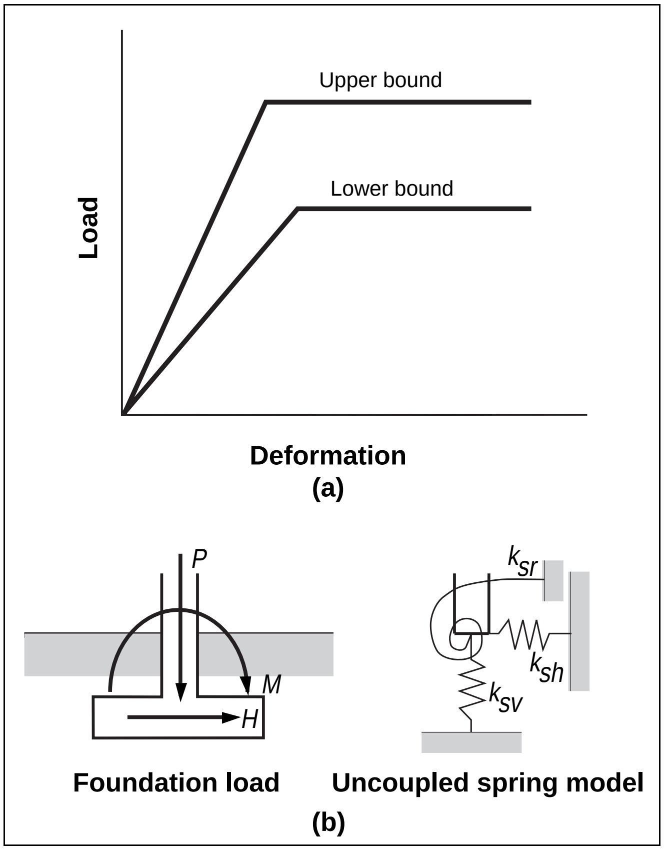


Figure 4-1 (a) Idealized Elasto-Plastic Load-Deformation Behavior for Soils (b) Uncoupled Spring Model for Rigid Footings

4.4.2.1 Shallow Bearing Foundations

A. Stiffness Parameters

The shear modulus, G, for a soil is related to the modulus of elasticity, E, and Poisson's ratio, ν, by the relationship

$$G = \frac{E}{2(1+\nu)} \tag{4-4}$$

Poisson's ratio may be assumed as 0.35 for unsaturated soils and 0.50 for saturated soils.

The initial shear modulus, G_o, is related to the shear wave velocity at low strains, v_s, and the mass density of the soil, ρ, by the relationship

$$G_o = \rho v_s^2 \tag{4-5}$$

(In the fonts currently in use in the *Guidelines*, the italicized v is similar to the Greek ν.) Converting mass density to unit weight, γ, gives an alternative expression

$$G_o = \frac{\gamma v_s^2}{g} \tag{4-6}$$

where g is acceleration due to gravity.

The initial shear modulus also has been related to normalized and corrected blow count, $(N_1)_{60}$, and effective vertical stress, σ'_o, as follows (from Seed et al., 1986):

$$G_o \cong 20{,}000\,(N_1)_{60}^{1/3}\sqrt{\sigma'_o} \tag{4-7}$$

where:

$(N_1)_{60}$	=	Blow count normalized for 1.0 ton per square foot confining pressure and 60% energy efficiency of hammer
σ'_o	=	Effective vertical stress in psf

and

σ'_o	=	$\gamma_t d - \gamma_w (d - d_w)$
γ_t	=	Total unit weight of soil
γ_w	=	Unit weight of water
d	=	Depth to sample
d_w	=	Depth to water level

It should be noted that the G_o in Equation 4-7 is expressed in pounds per square foot, as is σ'_o .

Most soils are intrinsically nonlinear and the shear wave modulus decreases with increasing shear strain. The large-strain shear wave velocity, v'_s , and the effective shear modulus, G, can be estimated based on the Effective Peak Acceleration coefficient for the earthquake under consideration, in accordance with Table 4-3.

Table 4-3 Effective Shear Modulus and Shear Wave Velocity

	Effective Peak Acceleration, $S_{XS}/2.5$	
	0.10	**0.70**
Ratio of effective to initial shear modulus (G/G_o)	0.50	0.20
Ratio of effective to initial shear wave velocity (v'_s/v_s)	0.71	0.45

Notes:

1. Site-specific values may be substituted if documented in a detailed geotechnical site investigation.
2. Linear interpolation may be used for intermediate values.

To reflect the upper and lower bound concept illustrated in Figure 4-1a in the absence of a detailed geotechnical site study, the upper bound stiffness of rectangular footings should be based on twice the effective shear modulus, G, determined in accordance with the above procedure. The lower bound stiffness should be based on one-half the effective shear modulus. Thus the range of stiffness should incorporate a factor of four from lower to upper bound.

Most shallow bearing footings are stiff relative to the soil upon which they rest. For simplified analyses, an uncoupled spring model, as shown in Figure 4-1b, may be sufficient. The three equivalent spring constants may be determined using conventional theoretical solutions for rigid plates resting on a semi-infinite elastic medium. Although frequency-dependent solutions are available, results are reasonably insensitive to loading frequencies within the range of parameters of interest for buildings subjected to earthquakes. It is sufficient to use static stiffnesses as representative of repeated loading conditions.

Figure 4-2 presents stiffness solutions for rectangular plates in terms of an equivalent circular radius.

Stiffnesses are adjusted for shape and depth using factors similar to those in Figure 4-3. Other formulations incorporating a wider range of variables may be found in Gazetas (1991). For the case of horizontal translation, the solution represents mobilization of base traction (friction) only. If the sides of the footing are in close contact with adjacent in situ foundation soil or well-compacted fill, significant additional stiffness may be assumed from passive pressure. A solution for passive pressure stiffness is presented in Figure 4-4.

For more complex analyses, a finite element representation of linear or nonlinear foundation behavior may be accomplished using Winkler component models. Distributed vertical stiffness properties may be calculated by dividing the total vertical stiffness by the area. Similarly, the uniformly distributed rotational stiffness can be calculated by dividing the total rotational stiffness of the footing by the moment of inertia of the footing in the direction of loading. In general, however, the uniformly distributed vertical and rotational stiffnesses are not equal. The two may be effectively decoupled for a Winkler model using a procedure similar to that illustrated in Figure 4-5. The ends of the rectangular footing are represented by end zones of relatively high stiffness over a length of approximately one-sixth of the footing width. The stiffness per unit length in these end zones is based on the vertical stiffness of a B x $B/6$ isolated footing. The stiffness per unit length in the middle zone is equivalent to that of an infinitely long strip footing.

In some instances, the stiffness of the structural components of the footing may be relatively flexible compared to the soil material; for example, a slender grade beam resting on stiff soil. Classical solutions for beams on elastic supports can provide guidance on when such effects are important. For example, a grade beam supporting point loads spaced at a distance of L might be considered flexible if:

$$\frac{EI}{L^4} < 10 k_{sv} B \tag{4-8}$$

where, for the grade beam,

E = Effective modulus of elasticity

I = Moment of inertia

B = Width

For most flexible foundation systems, the unit subgrade spring coefficient, k_{sv}, may be taken as

$$k_{sv} = \frac{1.3G}{B(1-\nu)} \tag{4-9}$$

B. Capacity Parameters

The specific capacity of shallow bearing foundations should be determined using fully plastic concepts and the generalized capacities of Section 4.4.1. Upper and lower bounds of capacities, as illustrated in Figure 4-1a, should be determined by multiplying the best estimate values by 2.0 and 0.5, respectively.

In the absence of moment loading, the vertical load capacity of a rectangular footing of width B and length L is

$$Q_c = q_c BL \tag{4-10}$$

For rigid footings subject to moment and vertical load, contact stresses become concentrated at footing edges, particularly as uplift occurs. The ultimate moment capacity, M_c, is dependent upon the ratio of the vertical load stress, q, to the vertical stress capacity, q_c. Assuming that contact stresses are proportional to vertical displacement and remain elastic up to the vertical stress capacity, q_c, it can be shown that uplift will occur prior to plastic yielding of the soil when q/q_c is less than 0.5. If q/q_c is greater than 0.5, then the soil at the toe will yield prior to uplift. This is illustrated in Figure 4-6. In general the moment capacity of a rectangular footing may be expressed as:

$$M_c = \frac{LP}{2}\left(1 - \frac{q}{q_c}\right) \tag{4-11}$$

where

P = Vertical load

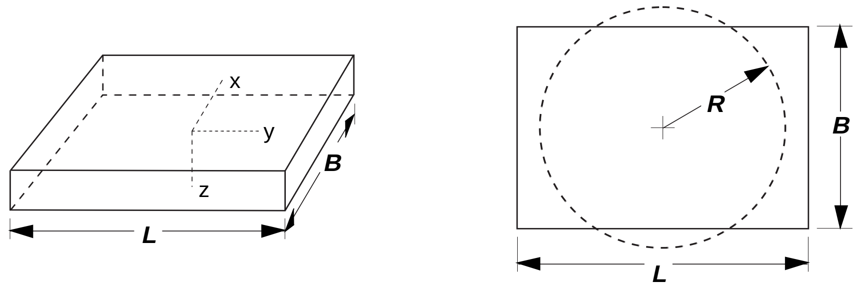
q = $\dfrac{P}{BL}$

B = Footing width

L = Footing length in direction of bending

Radii of circular footings equivalent to rectangular footings

Rectangular footing — *Equivalent circular footing*



	Degree of freedom			
	Translation	***Rocking***		***Torsion***
		About x-axis	*About y-axis*	*About z-axis*
Equivalent radius, R	$\left(\frac{BL}{\pi}\right)^{1/2}$	$\left(\frac{BL^3}{3\pi}\right)^{1/4}$	$\left(\frac{B^3L}{3\pi}\right)^{1/4}$	$\left[\frac{BL(B^2+L^2)}{6\pi}\right]^{1/4}$

Spring constants for embedded rectangular footings

Spring constants for shallow rectangular footings are obtained by modifying the solution for a circular footing, bonded to the surface of an elastic half-space, i.e., $k = \alpha\beta k_O$
where
k_O = Stiffness coefficient for the equivalent circular footing
α = Foundation shape correction factor (Figure 4-3a)
β = Embedment factor (Figure 4-3b)

To use the equation, the radius of an equivalent circular footing is first calculated according to the degree of freedom being considered. The figure above summarizes the appropriate radii. k_O is calculated using the table below:

Displacement degree of freedom	k_O
Vertical translation	$\frac{4GR}{1-\nu}$
Horizontal translation	$\frac{8GR}{2-\nu}$
Torsional rotation	$\frac{16GR^3}{3}$
Rocking rotation	$\frac{8GR^3}{3(1-\nu)}$

Note:
G and ν are the shear modulus and Poisson's ratio for the elastic half-space.
G is related to Young's modulus, E, as follows:
$E = 2(1+\nu)G$
R = Equivalent radius

Figure 4-2 Elastic Solutions for Rigid Footing Spring Constants (based on Gazetas, 1991 and Lam et al., 1991)

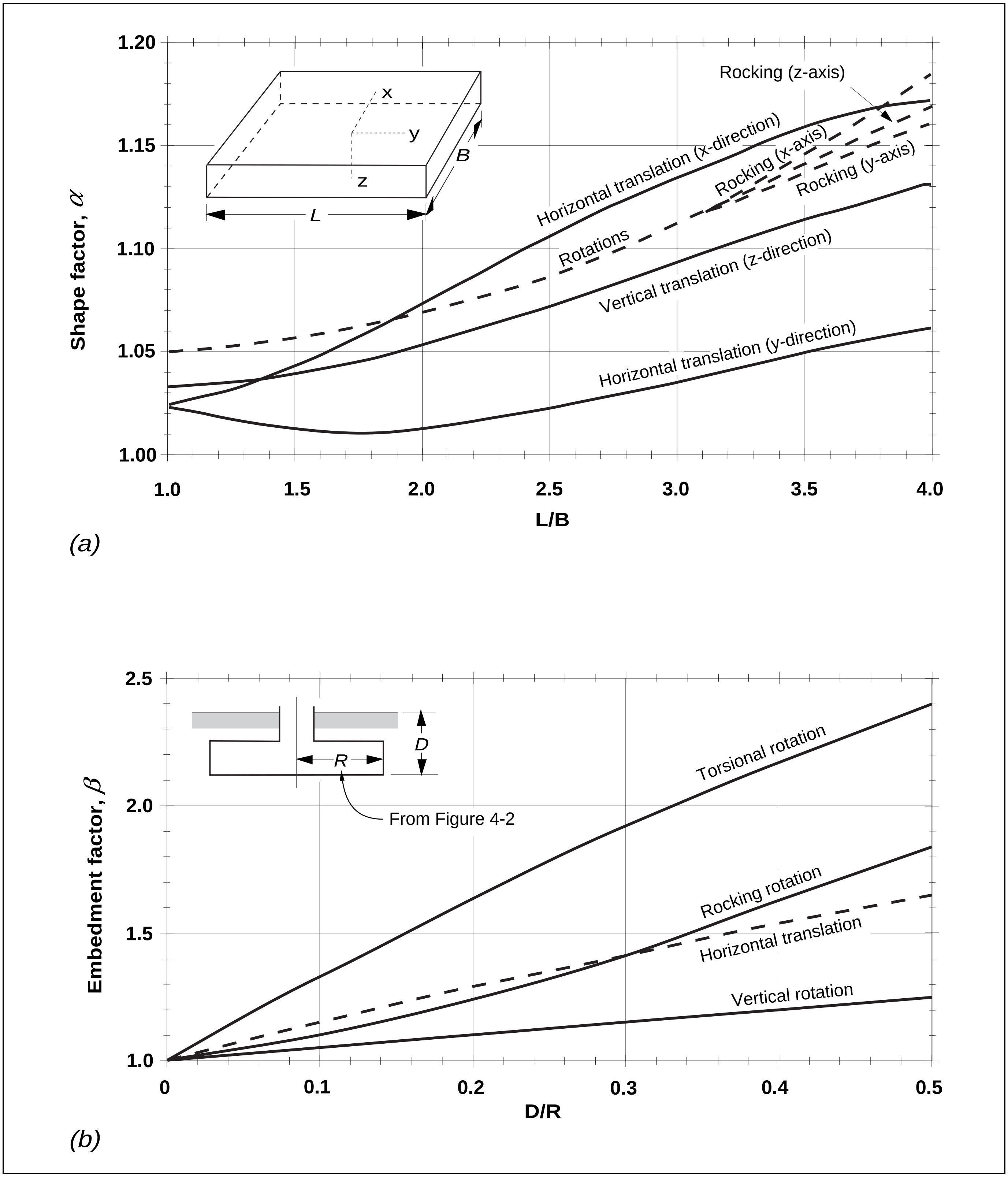


Figure 4-3 (a) Foundation Shape Correction Factors (b) Embedment Correction Factors

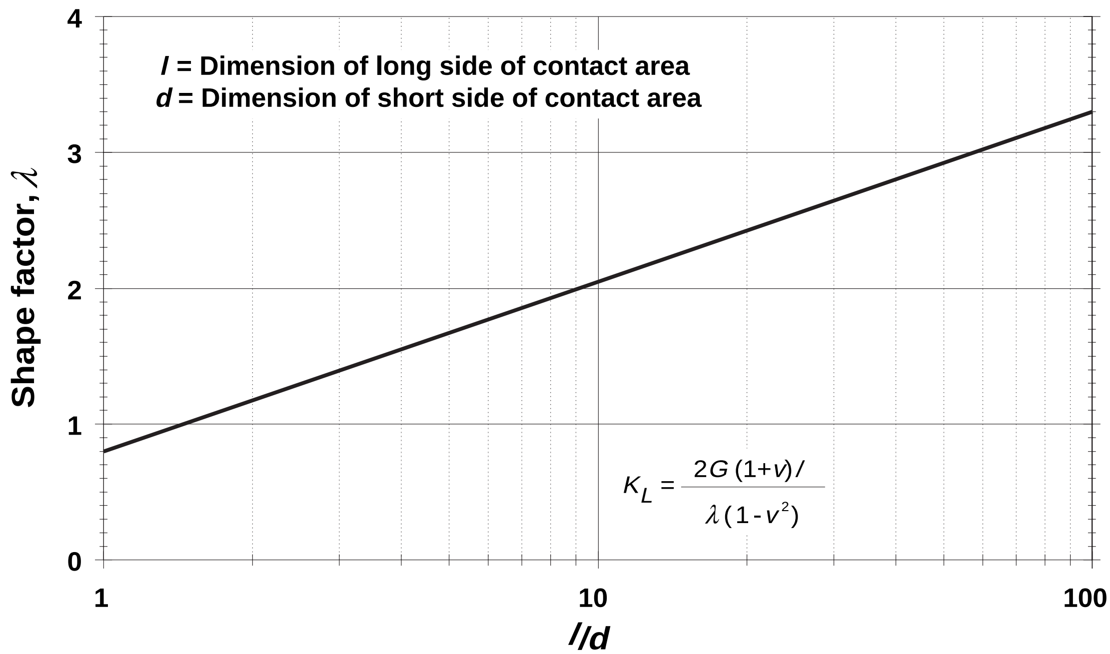


Figure 4-4 Lateral Foundation-to-Soil Stiffness for Passive Pressure (after Wilson, 1988)

The lateral capacity of a footing should assumed to be attained when the displacement, considering both base traction and passive pressure stiffnesses, reaches 2% of the thickness of the footing. Upper and lower bounds of twice and one-half of this value, respectively, also apply.

4.4.2.2 Pile Foundations

Pile foundations, in the context of this subsection, refer to those foundation systems that are composed of a pile cap and associated driven or cast-in-place piles, which together form a pile group. A single pile group may support a load-bearing column, or a linear sequence of pile groups may support a shear wall.

Generally, individual piles in a group could be expected to be less than two feet in diameter. The stiffness characteristics of single large-diameter piles or drilled shafts are described in Section 4.4.2.3.

A. Stiffness Parameters

For the purpose of simplified analyses, the uncoupled spring model as shown in Figure 4-1b may be used where the footing in the figure represents the pile cap. In the case of the vertical and rocking springs, it can be assumed that the contribution of the pile cap is relatively small compared to the contribution of the piles. In general, mobilization of passive pressures by either the pile caps or basement walls will control lateral spring stiffness. Hence, estimates of lateral spring stiffness can be computed using elastic solutions as described in Section 4.4.2.1A. In instances where piles may contribute significantly to lateral stiffness (i.e., very soft soils, battered piles), solutions using beam-column pile models are recommended.

Axial pile group stiffness spring values, k_{sv}, may be assumed to be in an upper and lower bound range, respectively, given by:

$$k_{sv} = \sum_{n=1}^{N} \frac{0.5\,A\,E}{L} \quad \text{to} \quad \sum_{n=1}^{N} \frac{2\,A\,E}{L} \tag{4-12}$$

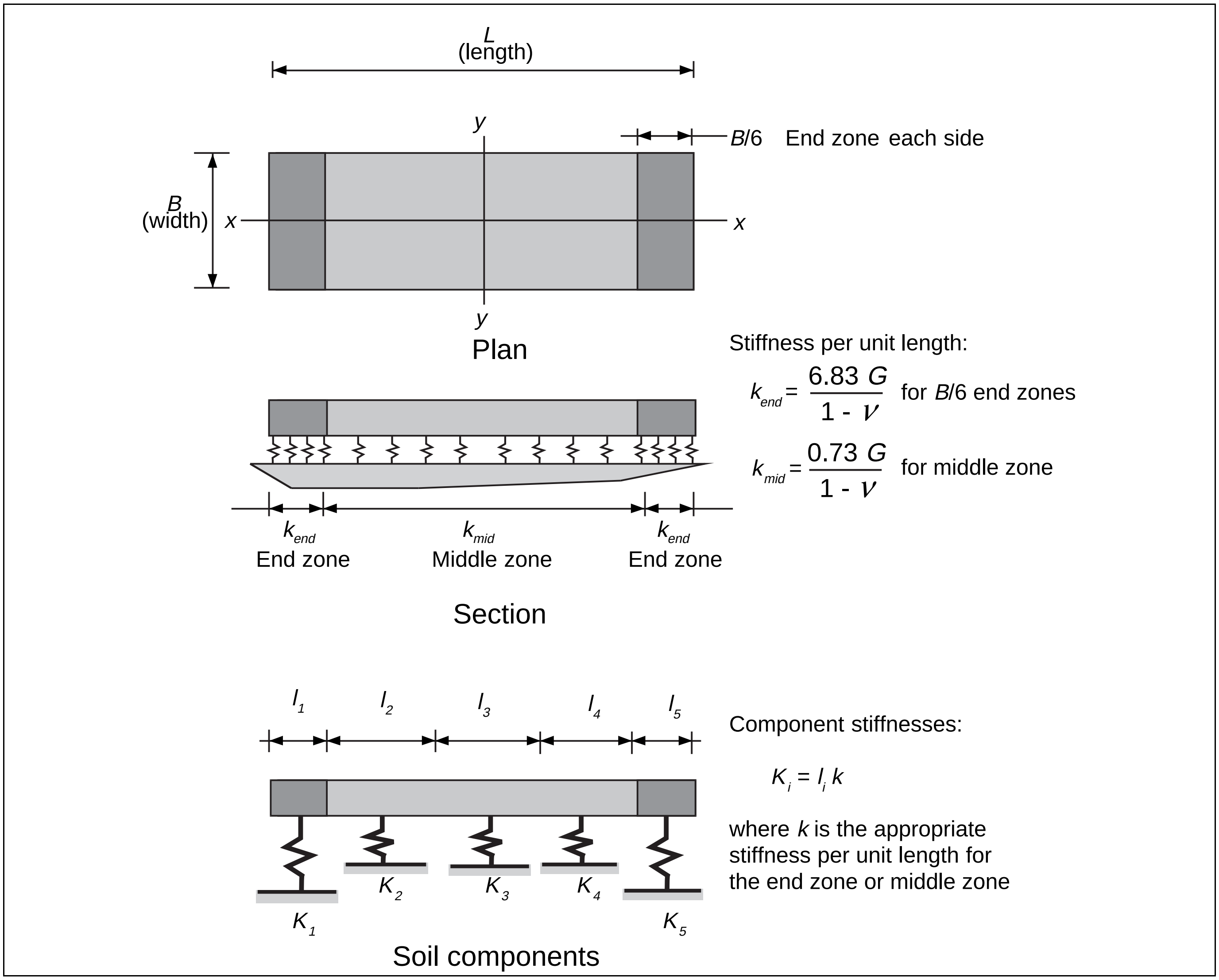


Figure 4-5 ***Vertical Stiffness Modeling for Shallow Bearing Footings***

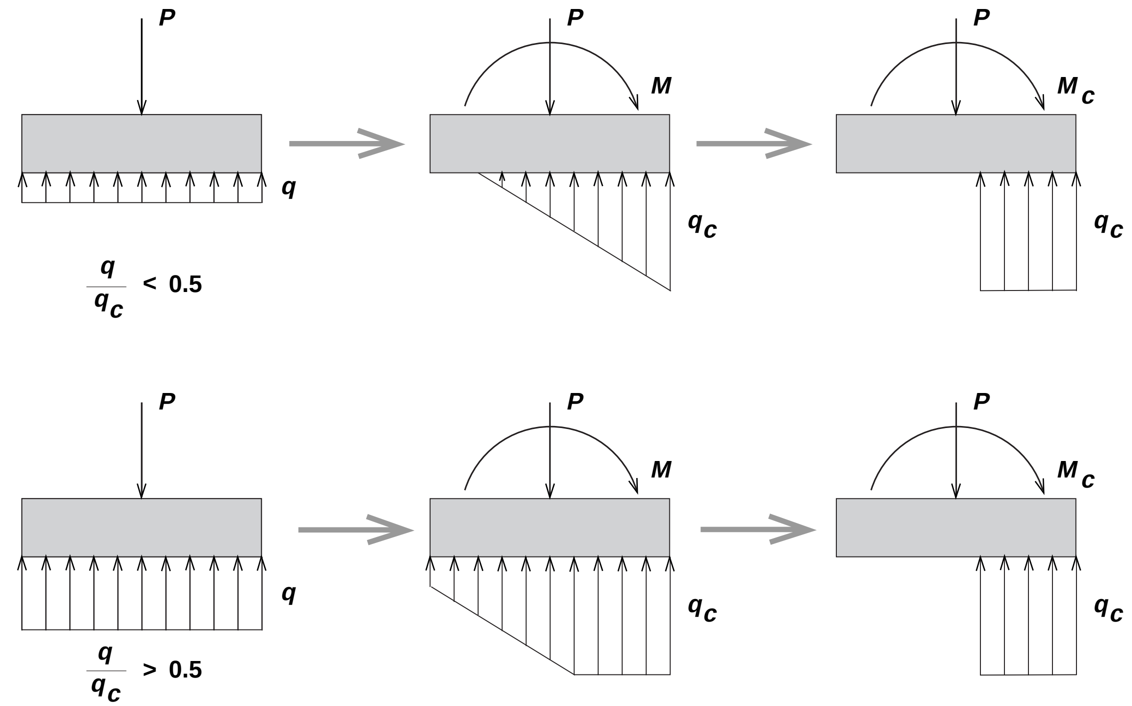


Figure 4-6 Idealized Concentration of Stress at Edge of Rigid Footings Subjected to Overturning Moment

where

A = Cross-sectional area of a pile

E = Modulus of elasticity of piles

L = Length of piles

N = Number of piles in group

The rocking spring stiffness values about each horizontal pile cap axis may be computed by assuming each axial pile spring acts as a discrete Winkler spring. The rotational spring constant (moment per unit rotation) is then given by:

$$k_{sr} = \sum_{n=1}^{N} k_{vn} S_n^2 \qquad (4\text{-}13)$$

where

k_{vn} = Axial stiffness of the nth pile

S_n = Distance between nth pile and axis of rotation

Whereas the effects of group action and the influence of pile batter are not directly accounted for in the form of the above equations, it can be reasonably assumed that the latter effects are accounted for in the range of uncertainties expressed for axial pile stiffness.

B. Capacity Parameters

Best-estimate vertical load capacity of piles (for both axial compression and axial tensile loading) should be determined using accepted foundation engineering practice, using best estimates of soil properties. Consideration should be given to the capability of pile cap and splice connections to take tensile loads when evaluating axial tensile load capacity. Upper and lower bound axial load capacities should be determined by multiplying best-estimate values by factors of 2.0 and 0.5, respectively.

The upper and lower bound moment capacity of a pile group should be determined assuming a rigid pile cap, leading to an initial triangular distribution of axial pile loading from applied seismic moments. However, full axial capacity of piles may be mobilized when computing ultimate moment capacity, leading to a rectangular distribution of resisting moment in a manner analogous to that described for a footing in Figure 4-6.

The lateral capacity of a pile group is largely dependent on that of the cap as it is restrained by passive resistance of the adjacent soil material. The capacity may be assumed to be reached when the displacement reaches 2% of the depth of the cap in a manner similar to that for a shallow bearing foundation.

4.4.2.3 Drilled Shafts

In general, drilled shaft foundations or piers may be treated similarly to pile foundations. When the diameter of the shaft becomes large (> 24 inches), the bending and the lateral stiffness and strength of the shaft itself may contribute to the overall capacity. This is obviously necessary for the case of individual shafts supporting isolated columns. In these instances, the interaction of the soil and shaft may be represented using Winkler type models (Pender, 1993; Reese et al., 1994).

4.4.3 Foundation Acceptability Criteria

This section contains acceptability criteria for the geotechnical components of building foundations. Structural components of foundations shall meet the appropriate requirements of Chapters 5 through 8.

Geotechnical components include the soil parts of shallow spread footings and mats, and friction- and end-bearing piles and piers. These criteria, summarized in Table 4-4, apply to all actions including vertical loads, moments, and lateral forces applied to the soil.

4.4.3.1 Simplified Rehabilitation

The geotechnical components of buildings qualified for and subject to Simplified Rehabilitation may be considered acceptable if they comply with the requirements of Chapter 10.

4.4.3.2 Linear Procedures

The acceptability of geotechnical components subject to linear procedures depends upon the basic modeling assumptions utilized in the analysis, as follows.

Fixed Base Assumption. If the base of the structure has been assumed to be completely rigid, actions on geotechnical components shall be as on force-controlled components governed by Equation 3-15 and component capacities may be assumed as upper-bound values. A fixed base assumption is not recommended for the Immediate Occupancy Performance Level for buildings sensitive to base rotations or other types of foundation movement.

Table 4-4 Soil Foundation Acceptability Summary

Analysis Procedure	Foundation Assumption	Performance Level	
		Collapse Prevention and Life Safety	Immediate Occupancy
Simplified Rehabilitation		See Chapter 10	Not applicable.
Linear Static or Dynamic	Fixed	Actions on geotechnical components shall be assumed as on force-controlled components governed by Equation 3-15 and component capacities may be assumed as upper bound values.	Not recommended for buildings sensitive to base rotation or other foundation movements.
	Flexible	$m = \infty$ for use in Equation 3-18	$m = 2.0$ for use in Equation 3-18
Nonlinear Static or Dynamic	Fixed	Base reactions limited to upper bound ultimate capacity.	Not recommended for buildings sensitive to base rotation or other foundation movements.
	Flexible	Geotechnical component displacements need not be limited, provided that structure can accommodate the displacements.	Estimate and accommodate possible permanent soil movements.

Flexible Base Assumption. If the base of the structure is modeled using linear geotechnical components, then the value of m, for use in Equation 3-18, for Life Safety and Collapse Prevention Performance Levels may be assumed as infinite, provided the resulting displacements may be accommodated within the acceptability criteria for the rest of the structure. For the

Immediate Occupancy Performance Levels, *m* values for geotechnical components shall be limited to 2.0.

4.4.3.3 Nonlinear Procedures

The acceptability of geotechnical components subject to nonlinear procedures depends upon the basic modeling assumptions utilized in the analysis, as follows.

Fixed Base Assumption. If the base of the structure has been assumed to be completely rigid, then the base reactions for all geotechnical components shall not exceed their upper-bound capacity to meet Collapse Prevention and Life Safety Performance Levels. A rigid base assumption is not recommended for the Immediate Occupancy Performance Level for buildings sensitive to base rotations or other types of foundation movement.

Flexible Base Assumption. If the base of the structure is modeled using flexible nonlinear geotechnical components, then the resulting component displacements need not be limited to meet Life Safety and Collapse Prevention Performance Levels, provided the resulting displacements may be accommodated within the acceptability criteria for the rest of the structure. For the Immediate Occupancy Performance Level, an estimate of the permanent nonrecoverable displacement of the geotechnical components shall be made based upon the maximum total displacement, foundation and soil type, soil layer thicknesses, and other pertinent factors. The acceptability of these displacements shall be based upon their effects on the continuing function and safety of the building.

4.5 Retaining Walls

Past earthquakes have not caused extensive damage to building walls below grade. In some cases, however, it may be advisable to verify the adequacy of retaining walls to resist increased pressure due to seismic loading. These situations might be for walls of poor construction quality, unreinforced or lightly reinforced walls, walls of archaic materials, unusually tall or thin walls, damaged walls, or other conditions implying a sensitivity to increased loads. The seismic earth pressure acting on a building wall retaining nonsaturated, level soil above the ground-water table may be approximated as:

$$\Delta p = 0.4 k_h \gamma_t H_{rw} \qquad (4\text{-}14)$$

where

Δp = Additional earth pressure due to seismic shaking, which is assumed to be a uniform pressure

k_h = Horizontal seismic coefficient in the soil, which may be assumed equal to $S_{XS}/2.5$

γ_t = The total unit weight of soil

H_{rw} = The height of the retaining wall

The seismic earth pressure given above should be added to the unfactored static earth pressure to obtain the total earth pressure on the wall. The expression in Equation 4-14 is a conservative approximation of the Mononabe-Okabe formulation. The pressure on walls during earthquakes is a complex action. If walls do not have the apparent capacity to resist the pressures estimated from the above approximate procedures, detailed investigation by a qualified geotechnical engineer is recommended.

4.6 Soil Foundation Rehabilitation

This section provides guidelines for modification to foundations to improve anticipated seismic performance. Specifically, the scope of this section includes suggested approaches to foundation modification and behavioral characteristics of foundation elements from a geotechnical perspective. These must be used in conjunction with appropriate structural material provisions from other chapters. Additionally, the acceptability of a modified structure is determined in accordance with Chapter 2 of the *Guidelines.*

4.6.1 Soil Material Improvements

Soil improvement options to increase the vertical bearing capacity of footing foundations are limited. Soil removal and replacement and soil vibratory densification usually are not feasible because they would induce settlements beneath the footings or be expensive to implement without causing settlement. Grouting may be considered to increase bearing capacity. Different grouting techniques are discussed in the *Commentary* Section C4.3.2. Compaction grouting can achieve densification and strengthening of a variety of soil types and/or extend foundation loads to deeper, stronger soils. The technique requires careful control to avoid causing uplift of foundation elements or adjacent floor slabs during the grouting process. Permeation grouting with chemical grouts can achieve substantial strengthening of sandy soils, but the more fine-grained or silty the sand, the less effective the technique becomes. Jet grouting could also be considered. These same techniques also may be considered to increase the lateral frictional resistance at the base of footings.

Options that can be considered to increase the passive resistance of soils adjacent to foundations or grade beams include removal and replacement of soils with stronger, well-compacted soils or with treated (e.g., cement-stabilized) soils; in-place mixing of soils with strengthening materials (e.g., cement); grouting, including permeation grouting and jet grouting; and in-place densification by impact or vibratory compaction (if the soil layers to be compacted are not too thick and vibration effects on the structure are tolerable).

4.6.2 Spread Footings and Mats

New isolated or spread footings may be added to existing structures to support new structural elements such as shear walls or frames. In these instances, capacities and stiffness may be determined in accordance with the procedures of Section 4.4.

Existing isolated or spread footings may be enlarged to increase bearing or uplift capacity. Generally, capacities and stiffness may be determined in accordance with the procedures of Section 4.4; however, consideration of existing contact pressures on the strength and stiffness of the modified footing may be required, unless a uniform distribution is achieved by shoring and/or jacking.

Existing isolated or spread footings may be underpinned to increase bearing or uplift capacity. This technique improves bearing capacity by lowering the contact horizon of the footing. Uplift capacity is improved by increasing the resisting soil mass above the footing. Generally, capacities and stiffness may be determined in accordance with the procedures of Section 4.4. Considerations of the effects of jacking and load transfer may be required.

Where potential for differential lateral displacement of building foundations exists, provision of interconnection with grade beams or a well-reinforced grade slab can provide good mitigation of these effects. Ties provided to withstand differential lateral displacement should have a strength based on rational analysis, with the advice of a geotechnical engineer when appropriate.

4.6.3 Piers and Piles

Piles and pile caps shall have the capacity to resist additional axial and shear loads caused by overturning forces. Wood piles cannot resist uplift unless a positive connection is provided for the loads. Piles must be reviewed for deterioration caused by decay, insect infestation, or other signs of distress.

Driven piles made of steel, concrete, or wood, or cast-in-place concrete piers may be used to support new structural elements such as shear walls or frames. Capacities and stiffnesses may be determined in accordance with the procedures of Section 4.4. When used in conjunction with existing spread footing foundations, the effects of differential foundation stiffness should be considered in the analysis of the modified structure.

Driven piles made of steel, concrete, or wood, or cast-in-place concrete piers may be used to supplement the vertical and lateral capacities of existing pile and pier foundation groups and of existing isolated and continuous spread footings. Capacities and stiffnesses may be determined in accordance with the procedures of Section 4.4. If existing loads are not redistributed by shoring and/or jacking, the potential for differential strengths and stiffnesses among individual piles or piers should be included.

4.7 Definitions

Allowable bearing capacity: Foundation load or stress commonly used in working-stress design (often controlled by long-term settlement rather than soil strength).

Deep foundation: Piles or piers.

Differential compaction: An earthquake-induced process in which loose or soft soils become more compact and settle in a nonuniform manner across a site.

Fault: Plane or zone along which earth materials on opposite sides have moved differentially in response to tectonic forces.

Footing: A structural component transferring the weight of a building to the foundation soils and resisting lateral loads.

Foundation soils: Soils supporting the foundation system and resisting vertical and lateral loads.

Foundation springs: Method of modeling to incorporate load-deformation characteristics of foundation soils.

Foundation system: Structural components (footings, piles).

Landslide: A down-slope mass movement of earth resulting from any cause.

Liquefaction: An earthquake-induced process in which saturated, loose, granular soils lose a substantial amount of shear strength as a result of increase in pore-water pressure during earthquake shaking.

Pier: Similar to pile; usually constructed of concrete and cast in place.

Pile: A deep structural component transferring the weight of a building to the foundation soils and resisting vertical and lateral loads; constructed of concrete, steel, or wood; usually driven into soft or loose soils.

Prescriptive ultimate bearing capacity: Assumption of ultimate bearing capacity based on properties prescribed in Section 4.4.1.2.

Presumptive ultimate bearing capacity: Assumption of ultimate bearing capacity based on allowable loads from original design.

Retaining wall: A free-standing wall that has soil on one side.

Shallow foundation: Isolated or continuous spread footings or mats.

SPT N-Values: Using a standard penetration test (ASTM Test D1586), the number of blows of a 140-pound hammer falling 30 inches required to drive a standard 2-inch-diameter sampler a distance of 12 inches.

Ultimate bearing capacity: Maximum possible foundation load or stress (strength); increase in deformation or strain results in no increase in load or stress.

4.8 Symbols

A	Footing area; also cross-section area of pile
B	Width of footing
D	Depth of footing bearing surface
E	Young's modulus of elasticity
G	Shear modulus
G_o	Initial or maximum shear modulus
H	Horizontal load on footing
H_{rw}	Height of retaining wall
I	Moment of inertia
K_L	Passive pressure stiffness
L	Length of footing in plan dimension
L	Length of pile in vertical dimension
M	Moment on footing
M_c	Ultimate moment capacity of footing
N	Number of piles in a pile group
$(N_1)_{60}$	Standard Penetration Test blow count normalized for an effective stress of 1 ton per square foot and corrected to an equivalent hammer energy efficiency of 60%
P	Vertical load on footing
Q_D	Dead (static) load
Q_E	Earthquake load
Q_L	Live (frequently applied) load
$Q_{allow.D+L}$	Allowable working dead plus live load for a pile as specified in original design documents
Q_c	Ultimate bearing capacity
Q_S	Snow load
R	Radius of equivalent circular footing
S_{XS}	Spectral response acceleration at short periods for any hazard level or damping, g
S_n	Distance between nth pile and axis of rotation of a pile group
S_S	Spectral response acceleration at short periods, obtained from response acceleration maps, g
c	Cohesive strength of soil, expressed in force/unit area (pounds/ft^2 or Pa)

d	Short side of footing lateral contact area
d	Depth to sample
d_w	Depth of ground-water level
g	Acceleration of gravity (386.1 in/sec.^{2}, or 9,800 mm/sec.^{2} for SI units)
k_h	Horizontal seismic coefficient in soil acting on retaining wall
k_o	Stiffness coefficient for equivalent circular footing
k_{sh}	Winkler spring coefficient in horizontal direction, expressed as force/unit displacement/unit area
k_{sr}	Winkler spring coefficient in overturning (rotation), expressed as force/unit displacement/unit area
k_{sv}	Winkler spring coefficient in vertical direction, expressed as force/unit displacement/unit area
k_{vn}	Axial stiffness of nth pile in a pile group
l	Long side of footing lateral contact area
m	A modification factor used in the acceptance criteria of deformation-controlled components or elements, indicating the available ductility of a component action, and a multiplier of ultimate foundation capacity for checking imposed foundation loads in Linear Static or Dynamic Procedures
q	Vertical bearing pressure
$q_{allow.D+L}$	Allowable working dead plus live load pressure for a spread footing as specified in original design documents
q_c	Ultimate bearing capacity
v_s	Shear wave velocity at low strain
v_s'	Shear wave velocity at high strain
Δp	Additional earth pressure on retaining wall due to seismic shaking
α	Foundation shape correction factor
β	Embedment factor
γ	Unit weight, weight/unit volume (pounds/ft^3 or N/m^3)
γ_t	Total unit weight of soil
γ_w	Unit weight of water
δ_c	Pile compliance
λ	Shape factor for lateral stiffness
ν	Poisson's ratio
ρ	Soil mass density
σ_o'	Effective vertical stress
ϕ	Angle of internal friction, degrees

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5. Steel and Cast Iron (Systematic Rehabilitation)

5.1 Scope

Rehabilitation measures for steel components and elements are described in this chapter. Information needed for systematic rehabilitation of steel buildings, as depicted in Step 4B of the Process Flow chart shown in Figure 1-1, is presented herein. A brief historical perspective is given in Section 5.2, with a more expanded version given in the *Commentary.*

Section 5.3 discusses material properties for new and existing construction, and describes material testing requirements for using the nonlinear procedures. A factor measuring the reliability of assumptions of in-place material properties is included in a kappa (κ) factor, used to account for accuracy of knowledge of the existing conditions. Evaluation methods for in-place materials are also described.

Sections 5.4 and 5.5 provide the attributes of steel moment frames and braced frames. The stiffness and strength properties of each steel component required for the linear and nonlinear procedures described in Chapter 3 are given. Stiffness and strength acceptance criteria are also given and are discussed within the context of Tables 2-1, 2-3, and 2-4, given in Chapter 2. These sections also provide guidance on choosing an appropriate rehabilitation strategy.

The appropriate procedures for evaluating systems with old and new components are discussed. Steel frames with concrete or masonry infills are briefly discussed, but the behavior of these systems and procedures for estimating the forces in the steel components are given in Chapters 6 (concrete) and 7 (masonry). Steel frames with attached masonry walls are discussed in this chapter and in Chapter 7.

Section 5.8 describes engineering properties for typical diaphragms found in steel buildings. These include bare metal deck, metal deck with composite concrete topping, noncomposite steel deck with concrete topping, horizontal steel bracing, and archaic diaphragms. The properties and behavior of wood diaphragms in steel buildings are presented in Chapter 8.

Engineering properties, and stiffness and strength acceptance criteria for steel piles are given in Section 5.9. Methods for calculating the forces in the piles are described in Chapter 4 and in the *Commentary* to Chapter 5.

5.2 Historical Perspective

The components of steel elements are columns, beams, braces, connections, link beams, and diaphragms. The columns, beams, and braces may be built up with plates, angles, and/or channels connected together with rivets, bolts, or welds. The material used in older construction is likely to be mild steel with a specified yield strength between 30 ksi and 36 ksi. Cast iron was often used for columns in much older construction (before 1900). Cast iron was gradually replaced by wrought iron and then steel. The connectors in older construction were usually mild steel rivets or bolts. These were later replaced by high-strength bolts and welds. The seismic performance of these components will depend heavily on the condition of the in-place material. A more detailed historical perspective is given in Section C5.2 of the *Commentary.*

As indicated in Chapter 1, great care should be exercised in selecting the appropriate rehabilitation approaches and techniques for application to historic buildings in order to preserve their unique characteristics.

5.3 Material Properties and Condition Assessment

5.3.1 General

Quantification of in-place material properties and verification of the existing system configuration and condition are necessary to analyze or evaluate a building. This section identifies properties requiring consideration and provides guidelines for their acquisition. Condition assessment is an important aspect of planning and executing seismic rehabilitation of an existing building. One of the most important steps in condition assessment is a visit to the building for visual inspection.

The extent of in-place materials testing and condition assessment that must be accomplished is related to availability and accuracy of construction and as-built records, the quality of materials used and construction performed, and the physical condition of the structure. Data such as the properties and grades of material used in component and connection fabrication may be effectively used to reduce the amount of in-place testing required. The design professional is encouraged to research and acquire all available records from original construction. The requirements given here are supplemental to those given in Section 2.7.

5.3.2 Properties of In-Place Materials and Components

5.3.2.1 Material Properties

Mechanical properties of component and connection material dictate the structural behavior of the component under load. Mechanical properties of greatest interest include the expected yield (F_{ye}) and tensile (F_{te}) strengths of base and connection material, modulus of elasticity, ductility, toughness, elogational characteristics, and weldability. The term “expected strength” is used throughout this document in place of “nominal strength” since expected yield and tensile stresses are used in place of nominal values specified in AISC (1994a and b).

The effort required to determine these properties is related to the availability of original and updated construction documents, original quality of construction, accessibility, and condition of materials.

The determination of material properties is best accomplished through removal of samples and laboratory testing. Sampling may take place in regions of reduced stress—such as flange tips at beam ends and external plate edges—to minimize the effects of reduced area. Types and sizes of specimens should be in accordance with ASTM standards. Mechanical and metallurgical properties usually can be established from laboratory testing on the same sample. If a connector such as a bolt or rivet is removed for testing, a comparable bolt should be reinstalled at the time of sampling. Destructive removal of a welded connection sample must be accompanied by repair of the connection.

5.3.2.2 Component Properties

Behavior of components, including beams, columns, and braces, is dictated by such properties as area, width-to-thickness and slenderness ratios, lateral torsional buckling resistance, and connection details. Component properties of interest are:

- Original cross-sectional shape and physical dimensions
- Size and thickness of additional connected materials, including cover plates, bracing, and stiffeners
- Existing cross-sectional area, section moduli, moments of inertia, and torsional properties at critical sections
- As-built configuration of intermediate, splice, and end connections
- Current physical condition of base metal and connector materials, including presence of deformation.

Each of these properties is needed to characterize building performance in the seismic analysis. The starting point for establishing component properties should be construction documents. Preliminary review of these documents shall be performed to identify primary vertical- and lateral-load-carrying elements and systems, and their critical components and connections. In the absence of a complete set of building drawings, the design professional must direct a testing agency to perform a thorough inspection of the building to identify these elements and components as indicated in Section 5.3.3.

In the absence of degradation, statistical analysis has shown that mean component cross-sectional dimensions are comparable to the nominal published values by AISC, AISI, and other organizations. Variance in these dimensions is also small.

5.3.2.3 Test Methods to Quantify Properties

To obtain the desired in-place mechanical properties of materials and components, it is necessary to utilize proven destructive and nondestructive testing methods. To achieve the desired accuracy, mechanical properties should be determined in the laboratory. Particular laboratory test information that may be sought includes yield and tensile strength, elongation, and charpy notch toughness. For each test, industry standards published by the ASTM exist and shall be followed. The *Commentary* provides applicability information and references for these particular tests.

Of greatest interest to metal building system performance are the expected yield and tensile strength of the installed materials. Notch toughness of structural steel and weld material is also important for connections that undergo cyclic loadings and deformations during earthquakes. Chemical and metallurgical properties can provide information on properties such as compatibility of welds with parent metal and potential lamellar tearing due to through-thickness stresses. Virtually all steel component elastic and inelastic limit states are related to yield and tensile strengths. Past research and accumulation of data by industry groups have resulted in published material mechanical properties for most primary metals and their date of fabrication. Section 5.3.2.5 provides this strength data. This information may be used, together with tests from recovered samples, to rapidly establish expected strength properties for use in component strength and deformation analyses.

Review of other properties derived from laboratory tests—such as hardness, impact, fracture, and fatigue—is generally not needed for steel component capacity determination, but is required for archaic materials and connection evaluation. These properties may not be needed in the analysis phase if significant rehabilitative measures are already known to be required.

To quantify material properties and analyze the performance of welded moment connections, more extensive sampling and testing may be necessary. This testing may include base and weld material chemical and metallurgical evaluation, expected strength determination, hardness, and charpy V-notch testing of the heat-affected zone and neighboring base metal, and other tests depending on connection configuration.

If any rehabilitative measures are needed and welded connection to existing components is required, the carbon equivalent of the existing component(s) shall be determined. Appropriate welding procedures are dependent upon the chemistry of base metal and filler material (for example, the elements in the IIW Carbon Equivalent formula). Consult Section 8 and its associated *Commentary* in the latest edition of ANSI/AWS D1.1 *Structural Welding Code*. Recommendations given in FEMA 267 (SAC, 1995) may also be followed.

5.3.2.4 Minimum Number of Tests

In order to quantify expected strength and other in-place properties accurately, it will sometimes be required that a minimum number of tests be conducted on representative components. As stated previously, the minimum number of tests is dictated by available data from original construction, the type of structural system employed, desired accuracy, and quality/condition of in-place materials. Access to the structural system will also be a factor in defining the testing program. As an alternative, the design professional may elect to utilize the default strength properties contained in Section 5.3.2.5 instead of the specified testing. However, in some cases these default values may only be used for a Linear Static Procedure (LSP).

Material properties of structural steel vary much less than those of other construction materials. In fact, the expected yield and tensile stresses are usually considerably higher than the nominal specified values. As a result, testing for material properties may not be required. The properties of wrought iron are more variable than those of steel. The strength of cast iron components cannot be determined from small sample tests, since component behavior is usually governed by inclusions and other imperfections. It is recommended that the lower-bound default value for compressive strength of cast iron given in Table 5-1 be used.

The guidelines for determining the expected yield (F_{ye}) and tensile (F_{te}) strengths are given below.

- If original construction documents defining properties—including material test records or material test reports (MTR)—exist, material tests need not be carried out, at the discretion of the design professional. Default values from Table 5-2 may be used. Larger values may be used, at the discretion of the design professional, if available historical data substantiates them. Larger values should be used if the assumptions produce a larger demand on associated connections.

- If original construction documents defining properties are limited or do not exist, but the date of construction is known and the single material used is confirmed to be carbon steel, at least three strength coupons shall be randomly removed from each component type. Conservative material properties such as those given in Table 5-2 may be used in lieu of testing, at the discretion of the design professional.

- If no knowledge exists of the structural system and materials used, at least two strength tensile coupons should be removed from each component type for every four floors. If it is determined from testing that more than one material grade exists, additional testing should be performed until the extent of use for each grade in component fabrication has been established. If it is determined that all components are made from steel, the requirements immediately preceding this may be followed.

- In the absence of construction records defining welding filler metals and processes used, at least one weld metal sample for each construction type should be obtained for laboratory testing. The sample shall consist of both local base and weld metal, such that composite strength of the connection can be derived. Steel and weld filler material properties discussed in Section 5.3.2.3 should also be obtained. Because of the destructive nature and necessary repairs that follow, default strength properties may be substituted if original records on welding exist, unless the design professional requires more accurate data. If ductility and toughness are required at or near the weld, the design professional may conservatively assume that no ductility is available, in lieu of testing. In this case the joint would have to be modified. Special requirements for welded moment frames are given in FEMA 267 (SAC, 1995) and the latest edition of ANSI/AWS D1.1 *Structural Welding Code*.

- Testing requirements for bolts and rivets are the same as for other steel components as given above. In lieu of testing, default values from Table 5-2 may be used.

- For archaic materials, including wrought iron but excluding cast iron, at least three strength coupons shall be extracted for each component type for every four floors of construction. Should significant variability be observed, in the judgment of the design professional, additional tests shall be performed until an acceptable strength value is obtained. If initial tests provide material properties that are consistent with properties given in Table 5-1, tests are required only for every six floors of construction.

For all laboratory test results, the mean yield and tensile strengths may be interpreted as the expected strength for component strength calculations.

For other material properties, the design professional shall determine the particular need for this type of testing and establish an adequate protocol consistent with that given above. In general, it is recommended that a minimum of three tests be conducted.

If a higher degree of confidence in results is desired, the sample size shall be determined using ASTM Standard E22 guidelines. Alternatively, the prior knowledge of material grades from Section 5.3.2.5 may be used in conjunction with Bayesian statistics to gain greater confidence with the reduced sample sizes noted above. The design professional is encouraged to use the procedures contained in the *Commentary* in this regard.

5.3.2.5 Default Properties

The default expected strength values for key metallic material properties are contained in Tables 5-1 and 5-2. These values are conservative, representing mean values from previous research less two standard deviations. It is recommended that the results of any material testing performed be compared to values in these tables for the particular era of building construction. Additional testing is recommended if the expected yield and tensile strengths determined from testing are lower than the default values.

Default material strength properties may only be used in conjunction with Linear Static and Dynamic Procedures. For the nonlinear procedures, expected strengths determined from the test program given above shall be used. Nonlinear procedures may be used with the reduced testing requirements described in *Commentary* Section C5.3.2.5.

5.3.3 Condition Assessment

5.3.3.1 General

A condition assessment of the existing building and site conditions shall be performed as part of the seismic rehabilitation process. The goals of this assessment are:

- To examine the physical condition of primary and secondary components and the presence of any degradation

- To verify or determine the presence and configuration of components and their connections, and the continuity of load paths between components, elements, and systems

Table 5-1 Default Material Properties[1]

Early unit stresses used in tables of allowable loads as published in catalogs of the following mills

Year	Rolling Mill	Expected Yield Strength, ksi
FOR CAST IRON[1]		
1873	Carnegie Kloman & Co. (“Factor of Safety 3”)	21
1874	New Jersey Steel & Iron Co.	18
1881–1884	Carnegie Brothers & Co., Ltd.	18 15
1884	The Passaic Rolling Mill Co.	18 15
1885	The Phoenix Iron Company	18
1885–1887	Pottsville Iron & Steel Co.	18
1889	Carnegie Phipps & Co., Ltd.	18 15
FOR STEEL[1]		
1887	Pottsville Iron & Steel Co.	23
1889–1893	Carnegie Phipps & Co., Ltd.	24
1893–1908	Jones & Laughlins Ltd. Jones & Laughlins Steel Co.	24 18
1896	Carnegie Steel Co., Ltd.	24
1897–1903	The Passaic Rolling Mills Co.	24 18
1898–1919	Cambria Steel Co.	24 18
1900–1903	Carnegie Steel Company	24
1907–1911	Bethlehem Steel Co.	24
1915	Lackawanna Steel Co.	24 18

1. Modified from unit stress values in AISC “Iron and Steel Beams from 1873 to 1952.”

- To review other conditions—such as neighboring party walls and buildings, the presence of nonstructural components, and limitations for rehabilitation—that may influence building performance
- To formulate a basis for selecting a knowledge factor (see Section 5.3.4).

The physical condition of existing components and elements, and their connections, must be examined for presence of degradation. Degradation may include environmental effects (e.g., corrosion, fire damage, chemical attack) or past/current loading effects (e.g., overload, damage from past earthquakes, fatigue, fracture). The condition assessment shall also examine for configurational problems observed in recent earthquakes, including effects of discontinuous components, improper welding, and poor fit-up.

Component orientation, plumbness, and physical dimensions should be confirmed during an assessment. Connections in steel components, elements, and systems require special consideration and evaluation. The load path for the system must be determined, and each connection in the load path(s) must be evaluated. This includes diaphragm-to-component and component-to-component connections. FEMA 267

Table 5-2 Default Expected Material Strengths [1]

History of ASTM and AISC Structural Steel Specification Stresses

Date	Specification	Remarks	ASTM Requirement	
			Expected Tensile Strength[2], F_{te}, ksi	**Expected Yield Strength[2, 3] F_{ye}, ksi**
1900	ASTM, A9	Rivet Steel	50	30
	Buildings	Medium Steel	60	35
1901–1908	ASTM, A9	Rivet Steel	50	1/2 T.S.
	Buildings	Medium Steel	60	1/2 T.S.
1909–1923	ASTM, A9	Structural Steel	55	1/2 T.S.
	Buildings	Rivet Steel	48	1/2 T.S.
1924–1931	ASTM, A7	Structural Steel	55	1/2 T.S. or not less than 30
		Rivet Steel	46	1/2 T.S. or not less than 25
	ASTM, A9	Structural Steel	55	1/2 T.S. or not less than 30
		Rivet Steel	46	1/2 T.S. or not less than 25
1932	ASTM, A140-32T issued as a tentative revision to ASTM, A9 (Buildings)	Plates, Shapes, Bars	60	1/2 T.S. or not less than 33
		Eyebar flats unannealed	67	1/2 T.S. or not less than 36
1933	ASTM, A140-32T discontinued and ASTM, A9 (Buildings) revised Oct. 30, 1933	Structural Steel	55	1/2 T.S. or not less than 30
	ASTM, A9 tentatively revised to ASTM, A9-33T (Buildings)	Structural Steel	60	1/2 T.S. or not less than 33
	ASTM, A141-32T adopted as a standard	Rivet Steel	52	1/2 T.S. or not less than 28
1934 on	ASTM, A9	Structural Steel	60	1/2 T.S. or not less than 33
	ASTM, A141	Rivet Steel	52	1/2 T.S. or not less than 28

1. Duplicated from AISC "Iron and Steel Beams 1873 to 1952."
2. Values shown in this table are based on mean minus two standard deviations and duplicated from "Statistical Analysis of Tensile Data for Wide-Flange Structural Shapes." The values have been reduced by 10%, since originals are from mill tests.
3. T.S. = Tensile strength

Table 5-2 Default Expected Material Strengths [1] (continued)

Additional default assumptions

Date	Specification	Remarks	Expected Tensile Strength[2], F_{te}, ksi	Expected Yield Strength[2, 3] F_{ye}, ksi
1961 on	ASTM, A36	Structural Steel		
	Group 1		54	37
	Group 2		52	35
	Group 3		52	32
	Group 4		53	30
	Group 5		61	35
	ASTM, A572, Grade 50	Structural Steel		
	Group 1		56	41
	Group 2		57	42
	Group 3		60	44
	Group 4		62	43
	Group 5		71	44
	Dual Grade	Structural Steel		
	Group 1		59	43
	Group 2		60	43
	Group 3		64	46
	Group 4		64	44

1. Duplicated from AISC “Iron and Steel Beams 1873 to 1952.”
2. Values shown in this table are based on mean minus two standard deviations and duplicated from “Statistical Analysis of Tensile Data for Wide-Flange Structural Shapes.” The values have been reduced by 10%, since originals are from mill tests.
3. T.S. = Tensile strength

(SAC, 1995) provides recommendations for inspection of welded steel moment frames.

The condition assessment also affords an opportunity to review other conditions that may influence steel elements and systems and overall building performance. Of particular importance is the identification of other elements and components that may contribute to or impair the performance of the steel system in question, including infills, neighboring buildings, and equipment attachments. Limitations posed by existing coverings, wall and ceiling space, infills, and other conditions shall also be defined such that prudent rehabilitation measures may be planned.

5.3.3.2 Scope and Procedures

The scope of a condition assessment shall include all primary structural elements and components involved in gravity and lateral load resistance. The degree of assessment performed also affects the κ factor that is used (see Section 5.3.4).

If coverings or other obstructions exist, indirect visual inspection through use of drilled holes and a fiberscope may be utilized. If this method is not appropriate, then local removal of covering materials will be necessary. The following guidelines shall be used.

- If detailed design drawings exist, exposure of at least one different primary connection shall occur for each connection type. If no deviations from the drawings exist, the sample may be considered representative. If deviations are noted, then removal of additional coverings from primary connections of that type must be done until the design professional has adequate knowledge to continue with the evaluation and rehabilitation.

- In the absence of construction drawings, the design professional shall establish inspection protocol that will provide adequate knowledge of the building needed for reliable evaluation and rehabilitation. For steel elements encased in concrete, it may be more cost effective to provide an entirely new lateral-load-resisting system.

Physical condition of components and connectors may also dictate the use of certain destructive and nondestructive test methods. If steel elements are covered by well-bonded fireproofing materials or encased in durable concrete, it is likely that their condition will be suitable. However, local removal of these materials at connections shall be performed as part of the assessment. The scope of this removal effort is dictated by the component and element design. For example, in a braced frame, exposure of several key connections may suffice if the physical condition is acceptable and configuration matches the design drawings. However, for moment frames it may be necessary to expose more connection points because of varying designs and the critical nature of the connections. See FEMA 267 (SAC, 1995) for inspection of welded moment frames.

5.3.3.3 Quantifying Results

The results of the condition assessment shall be used in the preparation of building system models in the evaluation of seismic performance. To aid in this effort, the results shall be quantified and reduced, with the following specific topics addressed:

- Component section properties and dimensions
- Connection configuration and presence of any eccentricities
- Type and location of column splices
- Interaction of nonstructural components and their involvement in lateral load resistance

The acceptance criteria for existing components depends on the design professional's knowledge of the condition of the structural system and material properties (as previously noted). All deviations noted between available construction records and as-built conditions shall be accounted for and considered in the structural analysis.

5.3.4 Knowledge (κ) Factor

As described in Section 2.7 and Tables 2-16 and 2-17, computation of component capacities and allowable deformations shall involve the use of a knowledge (κ) factor. For cases where a linear procedure will be used in the analysis, two categories of κ exist. This section further describes the requirements specific to metallic structural elements that must be accomplished in the selection of a κ factor.

A κ factor of 1.0 can be utilized when a thorough assessment is performed on the primary and secondary components and load path, and the requirements of Section 2.7 are met. The additional requirement for a κ factor of 1.0 is that the condition assessment be done in accordance with Section 5.3.3. In general, a κ factor of 1.0 may be used if the construction documents are available.

If the configuration and condition of an as-built component or connection are not adequately known (in the judgement of the design professional, because design documents are unavailable and it is deemed too costly to do a thorough condition assessment in accordance with Section 5.3.3), the κ factor used in the final component evaluation shall be reduced to 0.75. A κ factor of 0.75 shall be used for all cast and wrought iron components and their connectors. For encased components where construction documents are limited and knowledge of configuration and condition is incomplete, a factor of 0.75 shall be used. In addition, for steel moment and braced frames, the use of a κ factor of 0.75 shall occur when knowledge of connection details is incomplete. See also Section C2.7.2 in the *Commentary.*

5.4 Steel Moment Frames

5.4.1 General

Steel moment frames are those frames that develop their seismic resistance through bending of beams and columns and shearing of panel zones. Moment-resisting connections with calculable resistance are required between the members. The frames are categorized by the types of connection used and by the local and global stability of the members. Moment frames may act alone to resist seismic loads, or they may act in conjunction with concrete or masonry shear walls or braced steel frames to form a dual system. Special rules for design of new dual systems are included in AISC (1994a) and BSSC (1995).

Columns, beams, and connections are the components of moment frames. Beams and columns may be built-up members from plates, angles, and channels, cast or wrought iron segments, hot-rolled members, or cold-formed steel sections. Built-up members may be assembled by riveting, bolting, or welding. Connections between the members may be fully restrained (FR), partially restrained (PR), or nominally unrestrained (simple shear or pinned). The components may be bare steel, steel with a nonstructural coating for fire protection, or steel with either concrete or masonry encasement for fire protection.

Two types of frames are categorized in this document. Fully restrained (FR) moment frames are those frames for which no more than 5% of the lateral deflections arise from connection deformation. Partially restrained (PR) moment frames are those frames for which more than 5% of the lateral deflections result from connection deformation. In each case, the 5% value refers only to deflection due to beam-column deformation and not to frame deflections that result from column panel zone deformation.

5.4.2 Fully Restrained Moment Frames

5.4.2.1 General

Fully restrained (FR) moment frames are those moment frames with rigid connections. The connection shall be at least as strong as the weaker of the two members being joined. Connection deformation may contribute no more than 5% (not including panel zone deformation) to the total lateral deflection of the frame. If either of these conditions is not satisfied, the frame shall be characterized as partially restrained. The most common beam-to-column connection used in steel FR moment frames since the late 1950s required the beam flange to be welded to the column flange using complete joint penetration groove welds. Many of these connections have fractured during recent earthquakes. The design professional is referred to the *Commentary* and to FEMA 267 (SAC, 1995).

Fully restrained moment frames encompass both Special Moment Frames and Ordinary Moment Frames, defined in the Seismic Provisions for Structural Steel Buildings in Part 6 of AISC (1994a). These terms are not used in the *Guidelines*, but most of the requirements for these systems are reflected in AISC (1994a).

Requirements for general or seismic design of steel components given in AISC (1994a) or BSSC (1995) are to be followed unless superseded by provisions in these *Guidelines*. In all cases, the expected strength will be used in place of the nominal design strength by replacing F_y with F_{ye}.

5.4.2.2 Stiffness for Analysis

A. Linear Static and Dynamic Procedures

Axial area. This is the complete area of rolled or built-up shapes. For built-up sections, the effective area should be reduced if adequate load transfer mechanisms are not available. For elements fully encased in concrete, the stiffness may be calculated assuming full composite action if most of the concrete may be expected to remain after the earthquake. Composite action may not be assumed for strength unless adequate load transfer and ductility of the concrete can be assured.

Shear area. This is based on standard engineering procedures. The above comments, related to built-up sections, concrete encased elements, and composite action of floor beam and slab, apply.

Moment of inertia. The calculation of rotational stiffness of steel beams and columns in bare steel frames shall follow standard engineering procedures. For components encased in concrete, the stiffness shall include composite action, but the width of the composite section shall be taken as equal to the width of the flanges of the steel member and shall not include parts of the adjoining floor slab, unless there is an adequate and identifiable shear transfer mechanism between the concrete and the steel.

Joint Modeling. Panel zone stiffness may be considered in a frame analysis by adding a panel zone element to the program. The beam flexural stiffness may also be adjusted to account for panel zone stiffness or flexibility and the stiffness of the concrete encasement. Use center line analysis for other cases. Strengthened members shall be modeled similarly to existing members. The approximate procedure suggested for calculation of stiffness of PR moment frames given below may be used to model panel zone effects, if available computer programs cannot explicitly model panel zones.

Connections. The modeling of stiffness for connections for FR moment frames is not required since, by definition, the frame displacements are not significantly

(<5%) affected by connection deformation. The strength of the connection must be great enough to carry the expected moment strength and resulting shear in the beam at a beam-to-column connection and shall be calculated using standard engineering procedures. Three types of connections are currently acknowledged as potentially fully restrained: (1) full penetration (full-pen) welds between the flanges of the beam and column flanges with bolted or welded shear connections between the column flange and beam web; (2) flange plate connections; and (3) end plate connections. If flange plate or end plate connections are too flexible or weak to be considered fully restrained, they must be considered to be partially restrained. Strength and stiffness properties for these two connections as PR connections are discussed in Section 5.4.3 and in the *Commentary.*

B. Nonlinear Static Procedure

- Use elastic component properties as outlined under Section 5.4.2.2A.
- Use appropriate nonlinear moment-curvature and interaction relationships for beams and beam-columns to represent plastification. These may be derived from experiment or analysis.
- Linear and nonlinear behavior of panel zones shall be included.

In lieu of a more rational analysis, the details of all segments of the load-deformation curve, as defined in Tables 5-4 and Figure 5-1 (an approximate, generalized, load-deformation curve for components of steel moment frames, braced frames, and plate walls), may be used. This curve may be modified by assuming a strain-hardening slope of 3% of the elastic slope. Larger strain-hardening slopes may be used if verified by experiment. If panel zone yielding occurs, a strain-hardening slope of 6% or larger should be used for the panel zone. It is recommended that strain hardening be considered for all components.

The parameters Q and Q_{CE} in Figure 5-1 are generalized component load and generalized component expected strength for the component. For beams and columns, θ is the plastic rotation of the beam or column, θ_y is the rotation at yield, Δ is displacement, and Δ_y is yield displacement. For panel zones, θ_y is the angular shear deformation in radians. Figure 5-2 defines chord rotation for beams. The chord rotation may be estimated by adding the yield rotation, θ_y, to the plastic rotation. Alternatively, the chord rotation may be estimated to be equal to the story drift. Test results for steel components are often given in terms of chord rotation. The equations for θ_y given in Equations 5-1 and 5-2 are approximate, and are based on the assumption of a point of contraflexure at mid-length of the beam or column.

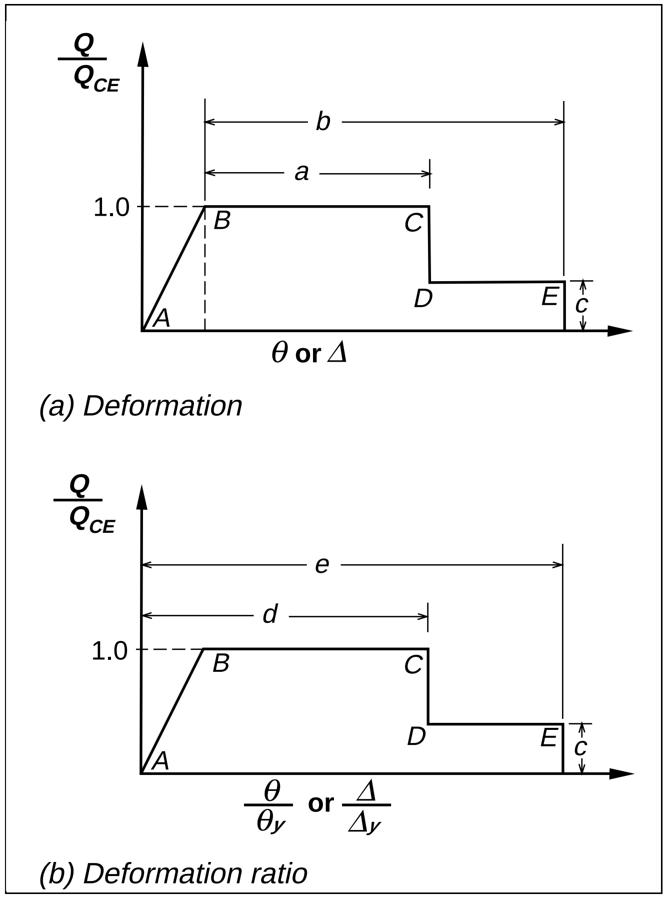

Figure 5-1 Definition of the a, b, c, d, and e Parameters in Tables 5-4, 5-6, and 5-8, and the Generalized Load-Deformation Behavior

Beams:
$$\theta_y = \frac{ZF_{ye}l_b}{6EI_b} \tag{5-1}$$

Columns:
$$\theta_y = \frac{ZF_{ye}l_c}{6EI_c}\left(1 - \frac{P}{P_{ye}}\right) \tag{5-2}$$

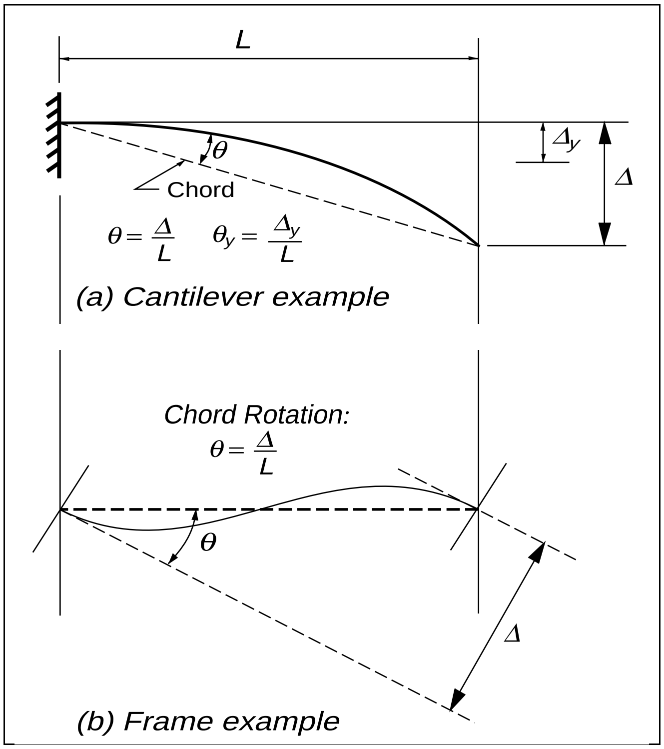

Figure 5-2 Definition of Chord Rotation

Q and Q_{CE} are the generalized component load and generalized component expected strength, respectively. For beams and columns, these refer to the plastic moment capacity, which is for:

Beams: $$Q_{CE} = M_{CE} = ZF_{ye} \quad (5\text{-}3)$$

Columns:

$$Q_{CE} = M_{CE} = 1.18ZF_{ye}\left(1 - \frac{P}{P_{ye}}\right) \leq ZF_{ye} \quad (5\text{-}4)$$

Panel Zones: $$Q_{CE} = V_{CE} = 0.55F_{ye}d_c t_p \quad (5\text{-}5)$$

where

d_c = Column depth, in.
E = Modulus of elasticity, ksi
F_{ye} = Expected yield strength of the material, ksi
I = Moment of inertia, in.^{4}
l_b = Beam length, in.
l_c = Column length, in.
M_{CE} = Expected moment strength
P = Axial force in the member, kips
P_{ye} = Expected axial yield force of the member = $A_g F_{ye}$, kips
Q = Generalized component load
Q_{CE} = Generalized component expected strength
t_p = Total panel zone thickness including doubler plates, in.
θ = Chord rotation
θ_y = Yield rotation
V_{CE} = Expected shear strength, kips
Z = Plastic section modulus, in.^{3}

C. Nonlinear Dynamic Procedure

The complete hysteretic behavior of each component must be properly modeled. This behavior must be verified by experiment. This procedure is not recommended in most cases.

5.4.2.3 Strength and Deformation Acceptance Criteria

A. Linear Static and Dynamic Procedures

The strength and deformation acceptance criteria for these methods require that the load and resistance relationships given in Equations 3-18 and 3-19 in Chapter 3 be satisfied. The design strength of components in existing FR moment frames shall be determined using the appropriate equations for design strength given in Section 5.4.2.2 or in Part 6 of AISC (1994a), except that ϕ shall be taken as 1.0. Design restrictions given in AISC (1994a) shall be followed unless specifically superseded by provisions in these *Guidelines*.

Evaluation of component acceptability requires knowledge of the component expected strength, Q_{CE}, for Equation 3-18 and the component lower-bound strength, Q_{CL}, for Equation 3-19, and the component demand modifier, m, as given in Table 5-3 for Equation 3-18. Values for Q_{CE} and Q_{CL} for FR moment frame components are given in this section. Q_{CE} and Q_{CL} are used for deformation- and force-controlled components, respectively. Values for m are given in Table 5-3 for the Immediate Occupancy, Life Safety, and Collapse Prevention Performance Levels.

Beams. The design strength of beams and other flexural members is the lowest value obtained according to the limit state of yielding, lateral-torsional buckling, local flange buckling, or shear yielding of the web. For fully concrete-encased beams where the concrete is expected to remain in place, because of confining reinforcement, during the earthquake, assume $b_f = 0$ and $L_p = 0$ for the purpose of determining m. For bare beams bent about their major axes and symmetric about both axes, with $\frac{b_f}{2t_f} < \frac{52}{\sqrt{F_{ye}}}$ (compact section) and $l_b < L_p$, the values for m are given in Table 5-3, and:

$$Q_{CE} = M_{CE} = M_{pCE} = ZF_{ye} \tag{5-6}$$

where

- b_f = Width of the compression flange, in.
- t_f = Thickness of the compression flange, in.
- l_b = Length of beam, in.
- L_p = Limiting lateral unbraced length for full plastic bending capacity for uniform bending from AISC (1994a), in.
- M_{CE} = Expected flexural strength, kip-in.
- M_{pCE} = Expected plastic moment capacity, kip-in.
- F_{ye} = Expected mean yield strength determined by the tests or given in Tables 5-1 or 5-2

If $\frac{b_f}{2t_f} > \frac{52}{\sqrt{F_y}}$ and $l_b > L_p$, values for m are given in Table 5-3. For cases where the moment diagram is nonuniform and $L_p < L_b < L_r$, but the nominal bending strength is still M_{pCE}, the value of m is obtained from Table 5-3. If $M_{CE} < M_{pCE}$ due to lateral torsional buckling, then the value of m shall be m_e, where

$$m_e = C_b\left[m - (m-1)\frac{(L_b - L_p)}{L_r - L_p}\right] \leq 8 \tag{5-7}$$

where

- L_b = Distance between points braced against lateral displacement of the compression flange, or between points braced to prevent twist of the cross section (see AISC, 1994a)
- L_p = Limiting unbraced length between points of lateral restraint for the full plastic moment capacity to be effective (see AISC, 1994a)
- L_r = Limiting unbraced length between points of lateral support beyond which elastic lateral torsional buckling of the beam is the failure mode (see AISC, 1994a)
- m = Value of m given in Table 5-3
- m_e = Effective m from Equation 5-7
- C_b = Coefficient to account for effect of nonuniform moment (see AISC, 1994a)

If the beam strength is governed by shear strength of the unstiffened web and $\frac{h}{t_w} \leq \frac{418}{\sqrt{F_y}}$, then:

$$Q_{CE} = V_{CE} = 0.6F_{ye}A_w \tag{5-8}$$

where

- V_{CE} = Expected shear strength, kips
- A_w = Nominal area of the web = $d_b t_w$, in.^{2}
- t_w = Web thickness, in.
- h = Distance from inside of compression flange to inside of tension flange, in.

For this case, use tabulated values for beams, row a, in Table 5-3. If $\frac{h}{t_w} > \frac{418}{\sqrt{F_y}}$, the value of V_{CE} should be calculated from provisions in Part 6 of AISC (1994a) and the value of m should be chosen using engineering judgment, but should be less than 8.

The limit state of local flange and lateral torsional buckling are not applicable to components either subjected to bending about their minor axes or fully encased in concrete, with confining reinforcement.

For built-up shapes, the strength may be governed by the strength of the lacing plates that carry component shear. For this case, the lacing plates are not as ductile as the component and should be designed for 0.5 times the *m* value in Table 5-3, unless larger values can be justified by tests or analysis. For built-up laced beams and columns fully encased in concrete, local buckling of the lacing is not a problem if most of the encasement can be expected to be in place after the earthquake.

Columns. The lower-bound strength, Q_{CL}, of steel columns under compression only is the lowest value obtained by the limit stress of buckling, local flange buckling, or local web buckling. The effective design strength should be calculated in accordance with provisions in Part 6 of AISC (1994a), but $\phi = 1.0$ and F_{ye} shall be used for existing components. Acceptance shall be governed by Equation 3-19 of these *Guidelines*, since this is a force-controlled member.

The lower-bound strength of cast iron columns shall be calculated as:

$$P_{CL} = A_g F_{cr} \tag{5-9}$$

where

$$F_{cr} = 12 \text{ ksi} \quad \text{for } l_c/r \le 108$$

$$F_{cr} = \frac{1.40 \times 10^5}{(l_c/r)^2} \text{ksi for } l_c/r > 108$$

Cast iron columns can only carry axial compression.

For steel columns under combined axial and bending stress, the column shall be considered to be deformation-controlled and the lower-bound strength shall be calculated by Equation 5-10 or 5-11.

For $\frac{P}{P_{CL}} \ge 0.2$

$$\frac{P}{P_{CL}} + \frac{8}{9}\left[\frac{M_x}{m_x M_{CEx}} + \frac{M_y}{m_y M_{CEy}}\right] \le 1.0 \tag{5-10}$$

For $\frac{P}{P_{CL}} < 0.2$

$$\frac{P}{2P_{CL}} + \frac{M_x}{m_x M_{CEx}} + \frac{M_y}{m_y M_{CEy}} \le 1.0 \tag{5-11}$$

where

P	=	Axial force in the column, kips
P_{CL}	=	Expected compression strength of the column, kips
M_x	=	Bending moment in the member for the x-axis, kip-in
M_{CEx}	=	Expected bending strength of the column for the x-axis, kip-in
M_{CEy}	=	Expected bending strength of the column for the y-axis
M_y	=	Bending moment in the member for the y-axis, kip-in
m_x	=	Value of *m* for the column bending about the x-axis
m_y	=	Value of *m* for the column bending about the y-axis

For columns under combined compression and bending, lateral bracing to prevent torsional buckling shall be provided as required by AISC (1994a).

Panel Zone. The strength of the panel zone shall be calculated as given in Equation 5-5.

Connections. By definition, the strength of FR connections shall be at least equal to, or preferably greater than, the strength of the members being joined. Some special considerations should be given to FR connections.

Full Penetration Welded Connections (Full-Pen). Full-pen connections (see Figure 5-3) have the beam flanges welded to the column flanges with complete penetration groove welds. A bolted or welded shear tab is also included to connect the beam web to the column. The strength and ductility of full-pen connections are not fully understood at this time. They are functions of the quality of construction, the l_b/d_b ratio of the beam (where l_b = beam length and d_b = beam depth), the weld material, the thickness of the beam and column flanges, the stiffness and strength of the panel zones, joint confinement, triaxial stresses, and other factors (see SAC, 1995). In lieu of further study, the value of *m* for Life Safety for beams with full-pen connections shall be not larger than

Table 5-3 Acceptance Criteria for Linear Procedures—Fully Restrained (FR) Moment Frames

Component/Action	*m* Values for Linear Procedures[8]				
	Primary			Secondary	
	IO m	LS m	CP m	LS m	CP m
Moment Frames					
Beams:					
a. $\frac{b}{2t_f} < \frac{52}{\sqrt{F_{ye}}}$	2	6	8	10	12
b. $\frac{b}{2t_f} > \frac{95}{\sqrt{F_{ye}}}$	1	2	3	3	4
c. For $\frac{52}{\sqrt{F_{ye}}} \le \frac{b}{2t_f} \le \frac{95}{\sqrt{F_{ye}}}$ use linear interpolation					
Columns:					
For $P/P_{ye} < 0.20$					
a. $\frac{b}{2t_f} < \frac{52}{\sqrt{F_{ye}}}$	2	6	8	10	12
b. $\frac{b}{2t_f} > \frac{95}{\sqrt{F_{ye}}}$	1	1	2	2	3
c. For $\frac{52}{\sqrt{F_{ye}}} \le \frac{b}{2t_f} \le \frac{95}{\sqrt{F_{ye}}}$ use linear interpolation					
For 0.2 ð P/P_{ye} ð 0.50[9]					
a. $\frac{b}{2t_f} < \frac{52}{\sqrt{F_{ye}}}$	1	—[1]	—[2]	—[3]	—[4]
b. $\frac{b}{2t_f} > \frac{95}{\sqrt{F_{ye}}}$	1	1	1.5	2	2
c. For $\frac{52}{\sqrt{F_{ye}}} \le \frac{b}{2t_f} \le \frac{95}{\sqrt{F_{ye}}}$ use linear interpolation					
Panel Zones	1.5	8	11	NA	NA
Fully Restrained Moment Connections[7]					
For full penetration flange welds and bolted or welded web connection: beam deformation limits					
a. No panel zone yield	1	—[5]	—[6]	3	4
b. Panel zone yield	0.8	2	2.5	2	2.5

1. $m = 9(1 - 1.7\,P/P_{ye})$
2. $m = 12(1 - 1.7\,P/P_{ye})$
3. $m = 15(1 - 1.7\,P/P_{ye})$
4. $m = 18(1 - 1.7\,P/P_{ye})$
5. $m = 6 - 0.125\,d_b$
6. $m = 7 - 0.125\,d_b$
7. If construction documents verify that notch-tough rated weldment was used, these values may be multiplied by two.
8. For built-up numbers where strength is governed by the facing plates, use one-half these *m* values.
9. If $P/P_{ye} > 0.5$, assume column to be force-controlled.

$$m = 6.0 - 0.125\ d_b \tag{5-12}$$

In addition, if the strength of the panel zone is less than 0.9 times the maximum shear force that can be delivered by the beams, then the *m* for the beam shall be

$$m = 2 \tag{5-13}$$

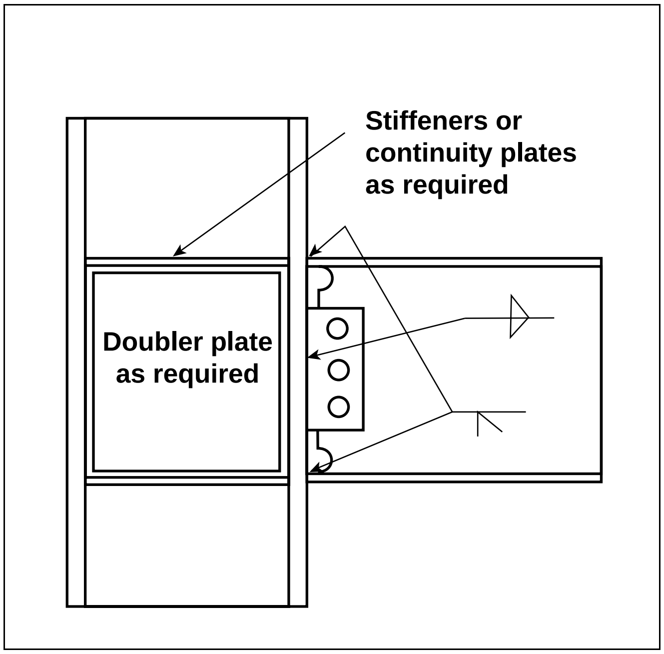

Figure 5-3 Full-Pen Connection in FR Connection with Variable Behavior

Flange Plate and End Plate Connections. The strength of these connections should be in accordance with standard practice as given in AISC (1994a and 1994b). Additional information for these connections is given below in Section 5.4.3.3.

Column Base Plates to Concrete Pile Caps or Footings. The strength of connections between column base plates and concrete pile caps or footings usually exceeds the strength of the columns. The strength of the base plate and its connection may be governed by the welds or bolts, the dimensions of the plate, or the expected yield strength, F_{ye}, of the base plate. The connection between the base plate and the concrete may be governed by shear or tension yield of the anchor bolts, loss of bond between the anchor bolts and the concrete, or failure of the concrete. Expected strengths for each failure type shall be calculated by rational analysis or the provisions in AISC (1994b). The values for *m* may be chosen from similar partially restrained end plate actions given in Table 5-5.

B. Nonlinear Static Procedure

The NSP requires modeling of the complete load-deformation relationship to failure for each component. This may be based on experiment, or on a rational analysis, preferably verified by experiment. In lieu of these, the conservative approximate behavior depicted by Figure 5-1 may be used. The values for Q_{CE} and θ_y shown in Figure 5-1 are the same as those used in the LSP and given in Section 5.4.2.2. Deformation control points and acceptance criteria for the Nonlinear Static and Dynamic Procedures are given in Table 5-4.

C. Nonlinear Dynamic Procedure

The complete hysteretic behavior of each component must be modeled for this procedure. Guidelines for this are given in the *Commentary.* Deformation limits are given in Table 5-4.

5.4.2.4 Rehabilitation Measures for FR Moment Frames

Several options are available for rehabilitation of FR moment frames. In all cases, the compatibility of new and existing components and/or elements must be checked at displacements consistent with the Performance Level chosen. The rehabilitation measures are as follows:

- Add steel braces to one or more bays of each story to form concentric or eccentric braced frames. (Attributes and design criteria for braced frames are given in Section 5.5.) Braces significantly increase the stiffness of steel frames. Care should be taken when designing the connections between the new braces and the existing frame. The connection should be designed to carry the maximum probable brace force, which may be approximated as 1.2 times the expected strength of the brace.

- Add ductile concrete or masonry shear walls or infill walls to one or more bays of each story. Attributes and design requirements of concrete and masonry infills are given in Sections 6.7 and 7.5, respectively. This greatly increases the stiffness and strength of the structure. Do not introduce torsional stress into the system.

Table 5-4 Modeling Parameters and Acceptance Criteria for Nonlinear Procedures—Fully Restrained (FR) Moment Frames

	$\frac{\Delta}{\Delta_y}$		Residual Strength Ratio	Plastic Rotation, Deformation Limits				
				Primary			Secondary	
Component/Action	***d***	***e***	***c***	**IO**	**LS**	**CP**	**LS**	**CP**
Beams[1]:								
a. $\frac{b}{2t_f} < \frac{52}{\sqrt{F_{ye}}}$	10	12	0.6	2	7	9	10	12
b. $\frac{b}{2t_f} > \frac{95}{\sqrt{F_{ye}}}$	5	7	0.2	1	3	4	4	5
c. For $\frac{52}{\sqrt{F_{ye}}} \leq \frac{b}{2t_f} \leq \frac{95}{\sqrt{F_{ye}}}$ use linear interpolation								
Columns[2]: For $P/P_{ye} < 0.20$								
a. $\frac{b}{2t_f} < \frac{52}{\sqrt{F_{ye}}}$	10	12	0.6	2	7	9	10	12
b. $\frac{b}{2t_f} > \frac{95}{\sqrt{F_{ye}}}$			0.2	1	3	4	4	5
c. For $\frac{52}{\sqrt{F_{ye}}} \leq \frac{b}{2t_f} \leq \frac{95}{\sqrt{F_{ye}}}$ use linear interpolation								

1. Add θ_y from Equations 5-1 or 5-2 to plastic end rotation to estimate chord rotation.
2. Columns in moment or braced frames need only be designed for the maximum force that can be delivered.
3. Deformation = 0.072 (1 – 1.7 P/P_{ye})
4. Deformation = 0.100 (1 – 1.7 P/P_{ye})
5. Deformation = 0.042 (1 – 1.7 P/P_{ye})
6. Deformation = 0.060 (1 – 1.7 P/P_{ye})
7. 0.043 – 0.0009 d_b
8. 0.035 – 0.0008 d_b
9. If $P/P_{ye} > 0.5$, assume column to be force-controlled.

Table 5-4 Modeling Parameters and Acceptance Criteria for Nonlinear Procedures—Fully Restrained (FR) Moment Frames (continued)

	$\frac{\Delta}{\Delta_y}$		Residual Strength Ratio	Plastic Rotation, Deformation Limits				
				Primary			Secondary	
Component/Action	***d***	***e***	***c***	**IO**	**LS**	**CP**	**LS**	**CP**
For $0.2 \le P/P_{ye} \le 0.50$[9]								
a. $\frac{b}{2t_f} < \frac{52}{\sqrt{F_{ye}}}$	—[3]	—[4]	0.2	0.04	—[5]	—[6]	0.019	0.031
b. $\frac{b}{2t_f} > \frac{95}{\sqrt{F_{ye}}}$	2	2.5	0.2	1	1.5	1.8	1.8	2
c. For $\frac{52}{\sqrt{F_{ye}}} \le \frac{b}{2t_f} \le \frac{95}{\sqrt{F_{ye}}}$ use linear interpolation								
	Plastic Rotation							
	a	**b**						
Panel Zones	0.052	0.081	0.800	0.004	0.025	0.043	0.055	0.067
Connections								
For full penetration flange weld, bolted or welded web: beam deformation limits								
a. No panel zone yield	—[7]	—[7]	0.200	0.008	—[8]	—[8]	0.017	0.025
b. Panel zone yield	0.009	0.017	0.400	0.003	0.005	0.007	0.010	0.013

1. Add θ_y from Equations 5-1 or 5-2 to plastic end rotation to estimate chord rotation.
2. Columns in moment or braced frames need only be designed for the maximum force that can be delivered.
3. Deformation = $0.072\ (1 - 1.7\ P/P_{ye})$
4. Deformation = $0.100\ (1 - 1.7\ P/P_{ye})$
5. Deformation = $0.042\ (1 - 1.7\ P/P_{ye})$
6. Deformation = $0.060\ (1 - 1.7\ P/P_{ye})$
7. $0.043 - 0.0009\ d_b$
8. $0.035 - 0.0008\ d_b$
9. If $P/P_{ye} > 0.5$, assume column to be force-controlled.

- Attach new steel frames to the exterior of the building. This scheme has been used in the past and has been shown to be very effective under certain conditions. Since this will change the distribution of stiffness in the building, the seismic load path must be carefully checked. The connections between the new and existing frames are particularly vulnerable. This approach may be structurally efficient, but it changes the architectural appearance of the building. The advantage is that the rehabilitation may take place without disrupting the use of the building.

- Reinforce the moment-resisting connections to force plastic hinge locations in the beam material away from the joint region. The idea behind this concept is that the stresses in the welded connection will be significantly reduced, thereby reducing the possibility of brittle fractures. This may not be effective if weld material with very low toughness was used in the full-pen connection. Strain hardening at the new hinge location may produce larger stresses at the weld than expected. Also, many fractures during past earthquakes are believed to have occurred at stresses lower than yield. Various methods, such as horizontal cover plates, vertical stiffeners, or haunches, can be employed. Other schemes that result in the removal of beam material may achieve the same purpose. Modification of all moment-resisting connections could significantly increase (or decrease, in the case of material removal) the structure's stiffness; therefore, recalculation of the seismic demands may be required. Modification of selected joints should be done in a rational manner that is justified by analysis. Guidance on the design of these modifications is discussed in SAC (1995).

- Adding damping devices may be a viable rehabilitation measure for FR frames. See Chapter 9 of these *Guidelines.*

5.4.3 Partially Restrained Moment Frames

5.4.3.1 General

Partially restrained (PR) moment frames are those frames for which deformation of the beam-to-column connections contributes greater than 5% of the story drift. A moment frame shall also be considered to be PR if the strength of the connections is less than the strength of the weaker of the two members being joined. A PR connection usually has two or more failure modes. The weakest failure mechanism shall be considered to govern the behavior of the joint. The beam and/or column need only resist the maximum force (or moment) that can be delivered by the connection. General design provisions for PR frames given in AISC (1994a) or BSSC (1995) shall apply unless superseded by these *Guidelines*. Equations for calculating nominal design strength shall be used for determining the expected strength, except $\phi = 1$, and F_{ye} shall be used in place of F_y.

5.4.3.2 Stiffness for Analysis

A. Linear Static and Dynamic Procedures

Beams, columns, and panel zones. Axial area, shear area, moment of inertia, and panel zone stiffness shall be determined as given in Section 5.4.2.2 for FR frames.

Connections. The rotational stiffness K_θ of each PR connection shall be determined by experiment or by rational analysis based on experimental results. The deformation of the connection shall be included when calculating frame displacements. Further discussion of this is given in the *Commentary*. In the absence of more rational analysis, the stiffness may be estimated by the following approximate procedures:

The rotational spring stiffness, K_θ, may be estimated by

$$K_\theta = \frac{M_{CE}}{0.005} \tag{5-14}$$

where

M_{CE} = Expected moment strength, kip-in.

for:

- PR connections that are encased in concrete for fire protection, and where the nominal resistance, M_{CE}, determined for the connection includes the composite action provided by the concrete encasement
- PR connections that are encased in masonry, where composite action cannot be developed in the connection resistance
- Bare steel PR connections

For all other PR connections, the rotational spring stiffness may be estimated by

$$K_\theta = \frac{M_{CE}}{0.003} \tag{5-15}$$

The connection strength, M_{CE}, is discussed in Section 5.4.3.3.

As a simplified alternative analysis method to an exact PR frame analysis, where connection stiffness is modeled explicitly, the beam stiffness, EI_b, may be adjusted by

$$EI_b \; adjusted = \frac{1}{\frac{6h}{l_b^2 K_\theta} + \frac{1}{EI_b}} \qquad (5\text{-}16)$$

where

K_θ = Equivalent rotational spring stiffness, kip-in./rad

M_{CE} = Expected moment strength, kip-in.

I_b = Moment of inertia of the beam, in.^{4}

h = Average story height of the columns, in.

l_b = Centerline span of the beam, in.

This adjusted beam stiffness may be used in standard rigid-connection frame finite element analysis. The joint rotation of the column shall be used as the joint rotation of the beam at the joint with this simplified analysis procedure.

B. Nonlinear Static Procedure

- Use elastic component properties as given in Section 5.4.3.2A.

- Use appropriate nonlinear moment-curvature or load-deformation behavior for beams, beam-columns, and panel zones as given in Section 5.4.2 for FR frames.

Use appropriate nonlinear moment-rotation behavior for PR connections as determined by experiment. In lieu of experiment, or more rational analytical procedure based on experiment, the moment-rotation relationship given in Figure 5-1 and Table 5-6 may be used. The parameters θ and θ_y are rotation and yield rotation. The value for θ_y may be assumed to be 0.003 or 0.005 in accordance with the provisions in Section 5.4.3.2A.

Q and Q_{CE} are the component moment and expected yield moment, respectively. Approximate values of M_{CE} for common types of PR connections are given in Section 5.4.3.3B.

C. Nonlinear Dynamic Procedure

The complete hysteretic behavior of each component must be properly modeled based on experiment.

5.4.3.3 Strength and Deformation Acceptance Criteria

A. Linear Static and Dynamic Procedures

The strength and deformation acceptance criteria for these methods require that the load and resistance relationships given in Equations 3-18 and 3-19 in Chapter 3 be satisfied. The expected strength and other restrictions for a beam or column shall be determined in accordance with the provisions given above in Section 5.4.2.3 for FR frames.

Evaluation of component acceptability requires knowledge of the lower-bound component capacity, Q_{CL} for Equation 3-19 and Q_{CE} for Equation 3-18, and the ductility factor, m, as given in Table 5-5 for use in Equation 3-18. Values for Q_{CE} and Q_{CL} for beams and columns in PR frames are the same as those given in Section 5.4.2.3 and Table 5-3 for FR frames. Values for Q_{CE} for PR connections are given in this section. Control points and acceptance criteria for Figure 5-1 for PR frames are given in Table 5-6. Values for m are given in Table 5-5 for the Immediate Occupancy, Life Safety, and Collapse Prevention Performance Levels.

B. Nonlinear Static Procedure

The NSP requires modeling of the complete load-deformation relationship to failure for each component. This may be based on experiment, or a rational analysis, preferably verified by experiment. In lieu of these, the conservative and approximate behavior depicted by Figure 5-1 may be used. The values for Q_{CE} and θ_y are the same as those used in the LSP in Sections 5.4.2.2 and 5.4.3.2. The deformation limits and nonlinear control points, c, d, and e, shown in Figure 5-1 are given in Table 5-6.

The expected strength, Q_{CE}, for PR connections shall be based on experiment or accepted methods of analysis as given in AISC (1994a and b) or in the *Commentary.* In lieu of these, approximate conservative expressions for Q_{CE} for common types of PR connections are given below.

Riveted or Bolted Clip Angle Connection. This is a beam-to-column connection as defined in Figure 5-4. The expected moment strength of the connection, M_{CE}, may be conservatively determined by using the smallest value of M_{CE} computed using Equations 5-17 through 5-22.

Table 5-5 Acceptance Criteria for Linear Procedures—Partially Restrained (PR) Moment Frames

Component/Action	*m* Values for Linear Methods				
	Primary			Secondary	
	IO	LS	CP	LS	CP
Partially restrained moment connection					
For top and bottom clip angles[1]					
a. Rivet or bolt shear failure[2]	1.5	4	6	6	8
b. Angle flexure failure	2	5	7	7	14
c. Bolt tension failure[2]	1	1.5	2.5	4	4
For top and bottom T-stub[1]					
a. Bolt shear failure[2]	1.5	4	6	6	8
b. T-stub flexure failure	2	5	7	7	14
c. Bolt tension failure[2]	1	1.5	2.5	4	4
For composite top and clip angle bottom[1]					
a. Yield and fracture of deck reinforcement	1	2	3	4	6
b. Local yield and web crippling of column flange	1.5	4	6	5	7
c. Yield of bottom flange angle	1.5	4	6	6	7
d. Tensile yield of column connectors or OSL of angle	1	1.5	2.5	2.5	3.5
e. Shear yield of beam flange connections	1	2.5	3.5	3.5	4.5
For flange plates welded to column bolted or welded to beam[1]					
a. Failure in net section of flange plate or shear failure of bolts or rivets[2]	1.5	4	5	4	5
b. Weld failure or tension failure on gross section of plate	0.5	1.5	2	1.5	2
For end plate welded to beam bolted to column					
a. Yielding of end plate	2	5.5	7	7	7
b. Yield of bolts	1.5	2	3	4	4
c. Failure of weld	0.5	1.5	2	3	3

1. Assumed to have web plate or stiffened seat to carry shear. Without shear connection, this may not be downgraded to a secondary member. If $d_b >$ 18 inches, multiply m values by $18/d_b$.
2. For high-strength bolts, divide these values by two.

If the shear connectors between the beam flange and the flange angle control the resistance of the connection:

$$Q_{CE} = M_{CE} = d_b(F_{ve}A_bN_b) \tag{5-17}$$

where

A_b = Gross area of rivet or bolt, in.^{2}

d_b = Overall beam depth, in.

F_{ve} = Unfactored nominal shear strength of the bolts or rivets given in AISC (1994a), ksi

N_b = Least number of bolts or rivets connecting the top or bottom flange to the angle

If the tensile capacity of the horizontal outstanding leg (OSL) of the connection controls the capacity, then P_{CE} is the smaller of

$$P_{CE} \le F_{ye}A_g \tag{5-18}$$

Table 5-6 Modeling Parameters and Acceptance Criteria for Nonlinear Procedures—Partially Restrained (PR) Moment Frames

	Plastic Rotation[1]		Residual Force Ratio	Joint Rotation				
				Primary			Secondary	
	a	*b*	*c*	IO	LS	CP	LS	CP
Top and Bottom Clip Angles[1]								
a. Rivet or bolt shear[2]	0.036	0.048	0.200	0.008	0.020	0.030	0.030	0.040
b. Angle flexure	0.042	0.084	0.200	0.010	0.025	0.035	0.035	0.070
c. Bolt tension	0.016	0.025	1.000	0.005	0.008	0.013	0.020	0.020
Top and Bottom T-Stub[1]								
a. Rivet or bolt shear[2]	0.036	0.048	0.200	0.008	0.020	0.030	0.030	0.040
b. T-stub flexure	0.042	0.084	0.200	0.010	0.025	0.035	0.035	0.070
c. Rivet or bolt tension	0.016	0.024	0.800	0.005	0.008	0.013	0.020	0.020
Composite Top Angle Bottom[1]								
a. Deck reinforcement	0.018	0.035	0.800	0.005	0.010	0.015	0.020	0.030
b. Local yield column flange	0.036	0.042	0.400	0.008	0.020	0.030	0.025	0.035
c. Bottom angle yield	0.036	0.042	0.200	0.008	0.020	0.030	0.025	0.035
d. Connectors in tension	0.015	0.022	0.800	0.005	0.008	0.013	0.013	0.018
e. Connections in shear[2]	0.022	0.027	0.200	0.005	0.013	0.018	0.018	0.023
Flange Plates Welded to Column Bolted or Welded to Beam[2]								
a. Flange plate net section or shear in connectors	0.030	0.030	0.800	0.008	0.020	0.025	0.020	0.025
b. Weld or connector tension	0.012	0.018	0.800	0.003	0.008	0.010	0.010	0.015
End Plate Bolted to Column Welded to Beam								
a. End plate yield	0.042	0.042	0.800	0.010	0.028	0.035	0.035	0.035
b. Yield of bolts	0.018	0.024	0.800	0.008	0.010	0.015	0.020	0.020
c. Fracture of weld	0.012	0.018	0.800	0.003	0.008	0.010	0.015	0.015

1. If $d_b > 18$, multiply deformations by $18/d_b$. Assumed to have web plate to carry shear. Without shear connection, this may not be downgraded to a secondary member.
2. For high-strength bolts, divide rotations by 2.

$$P_{CE} \le F_{te} A_e \qquad (5\text{-}19)$$

and

$$Q_{CE} = M_{CE} \le P_{CE}(d_b + t_a) \qquad (5\text{-}20)$$

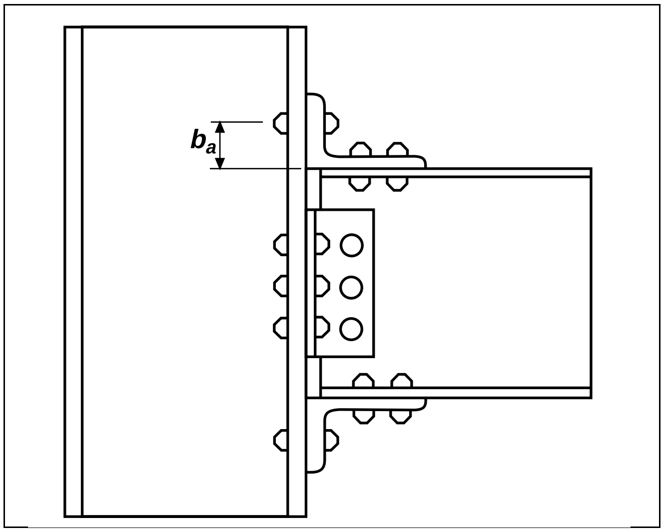

Figure 5-4 Clip Angle Connection

where

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

A_g = Gross area of the OSL, in.^{2}

P = Force in the OSL, kips

t_a = Thickness of angle, in.

If the tensile capacity of the rivets or bolts attaching the OSL to the column flange control the capacity of the connection:

$$Q_{CE} = M_{CE} = (d_b + b_a)(F_{te} A_c N_b) \qquad (5\text{-}21)$$

where

A_c = Rivet or bolt area, in.^{2}

b_a = Dimension in Figure 5-4, in.

F_{te} = Expected tensile strength of the bolts or rivets, ksi

N_b = Least number of bolts or rivets connecting top or bottom angle to column flange

Flexural yielding of the flange angles controls the expected strength if:

$$Q_{CE} = M_{CE} = \frac{w t_a^2 F_{ye}}{4\left[b_a - \frac{t_a}{2}\right]}(d_b + b_a) \qquad (5\text{-}22)$$

where

b_a = Dimension shown in Figure 5-4, in.

w = Length of the flange angle, in.

F_{ye} = Expected yield strength

Riveted or Bolted T-Stub Connection. A riveted or bolted T-stub connection is a beam-to-column connection as depicted in Figure 5-5. The expected moment strength, M_{CE}, may be determined by using the smallest value of M_{CE} computed using Equations 5-23 through 5-25.

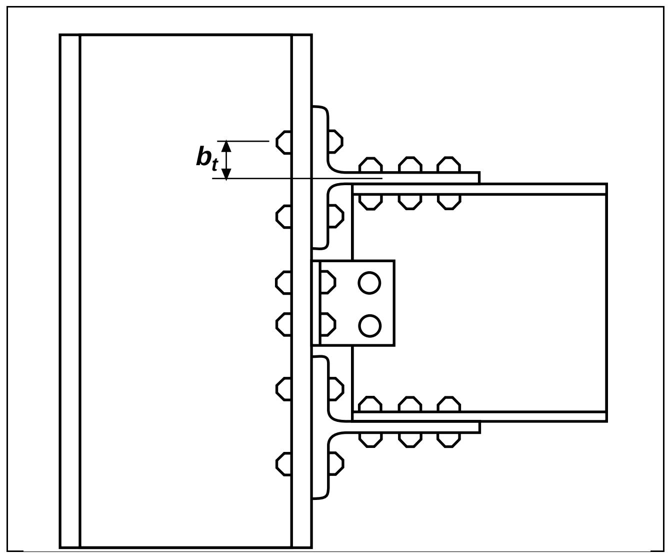

Figure 5-5 T-Stub Connection may be FR or PR Connection

If the shear connectors between the beam flange and the T-stub web control the resistance of the connection, use Equation 5-17.

If the tension capacity of the bolts or rivets connecting the T-stub flange to the column flange control the resistance of the connection:

$$Q_{CE} = M_{CE} = (d_b + 2b_t + t_s)(F_{te} A_b N_b) \qquad (5\text{-}23)$$

where

N_b = Number of fasteners in tension connecting the flanges of one T-stub to the column flange

t_s = Thickness of T-stub stem

If tension in the stem of the T-stub controls the resistance, use Equations 5-18 and 5-19 with A_g and A_e being the gross and net areas of the T-stub stem.

If flexural yielding of the flanges of the T-stub controls the resistance of the connection:

$$Q_{CE} = M_{CE} = \frac{(d_b + t_s)wt_f^2 F_{ye}}{2(b_t - k_1)} \qquad (5\text{-}24)$$

where

k_1 = Distance from the center of the T-stub stem to the edge of the T-stub flange fillet, in.

b_t = Distance between one row of fasteners in the T-stub flange and the centerline of the stem (Figure 5-5; different from b_a in Figure 5-4)

w = Length of T-stub, in.

t_f = Thickness of T-stub flange, in.

Flange Plate Connections. Flange plate connections are sometimes used as shown in Figure 5-6. This connection may be considered to be fully restrained if the strength is sufficient to develop the strength of the beam. The expected strength of the connection may be calculated as

$$Q_{CE} = M_{CE} = P_{CE}(d_b + t_p) \qquad (5\text{-}25)$$

where

P_{CE} = Expected strength of the flange plate connection as governed by the net section of the flange plate or the shear capacity of the bolts or welds, kips

t_p = Thickness of flange plate, in.

The strength of the welds must also be checked. The flange plates may also be bolted to the beam; in this case, the strength of the bolts and the net section of the flange plates must also be checked.

End Plate Connections. As shown in Figure 5-7, these may sometimes be considered to be FR if the strength is great enough to develop the expected strength of the beam. The strength may be governed by the bolts that are under combined shear and tension or bending in the end plate. The design strength $Q_{CE} = M_{CE}$ shall be computed in accordance with AISC (1994b) or by any other rational procedure supported by experimental results.

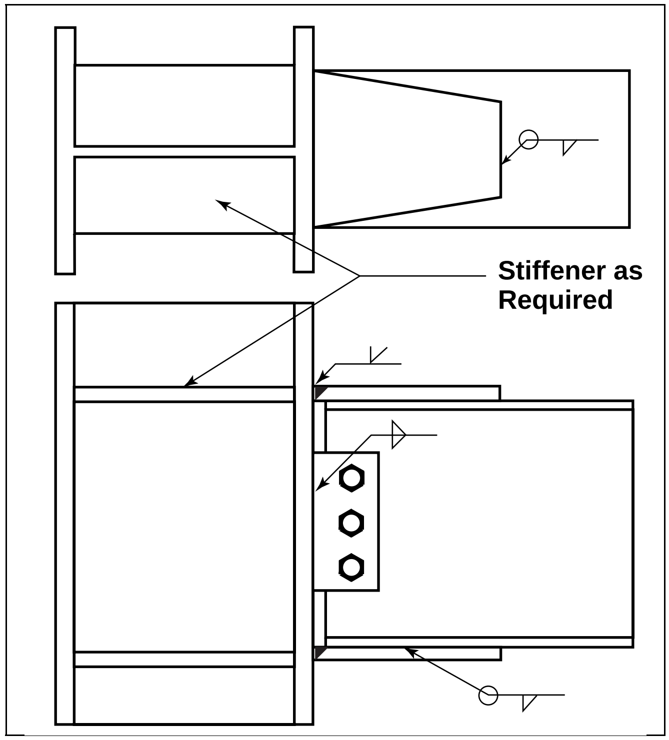

Figure 5-6 Flange Plate Connection may be FR or PR Connection

Composite Partially Restrained Connections. These may be used as shown in Figure 5-8. The equivalent rotational spring constant, K_θ, shall be that given by Equation 5-14. The behavior of these connections is complex, with several possible failure mechanisms. Strength calculations are discussed in the *Commentary.*

C. Nonlinear Dynamic Procedure

See Section 5.4.2.3.

5.4.3.4 Rehabilitation Measures for PR Moment Frames

The rehabilitation measures for FR moment frames will often work for PR moment frames as well (see Section 5.4.2.4). PR moment frames are often too flexible to provide adequate seismic performance. Adding concentric or eccentric bracing, or reinforced concrete or masonry infills, may be a cost-effective rehabilitation measure.

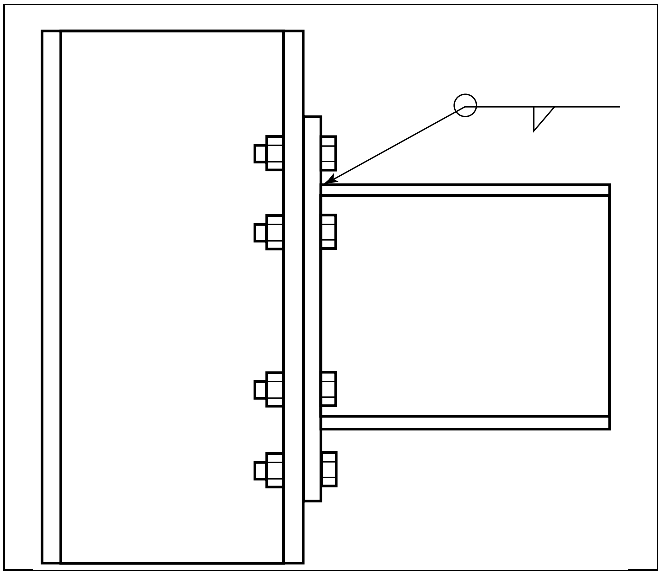

Figure 5-7 End Plate Connection may be FR or PR Connection

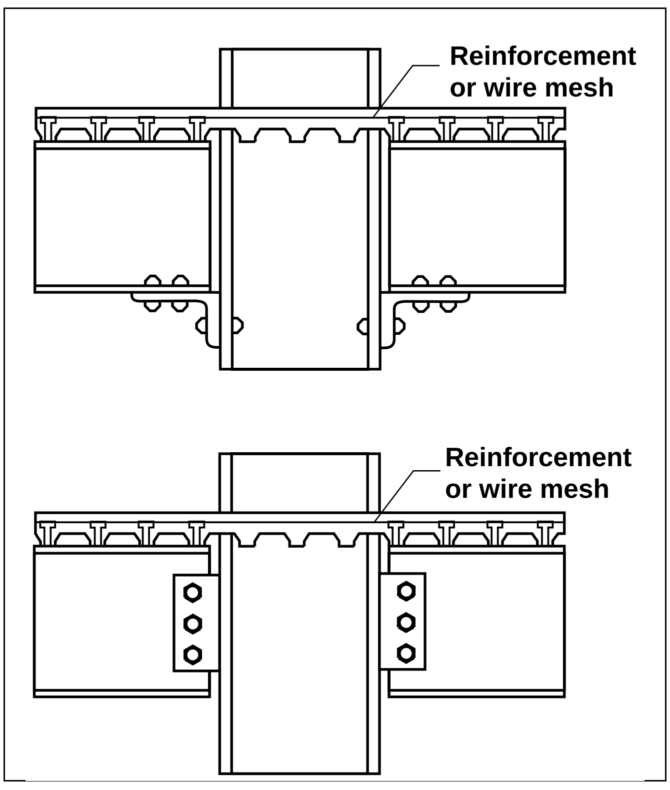

Figure 5-8 Two Configurations of PR Composite Connections

Connections in PR moment frames are usually the weak, flexible, or both, components. Connections may be rehabilitated by replacing rivets with high-strength bolts, adding weldment to supplement rivets or bolts, welding stiffeners to connection pieces or combinations of these measures.

5.5 Steel Braced Frames

5.5.1 General

The seismic resistance of steel braced frames is primarily derived from the axial force capacity of their components. Steel braced frames act as vertical trusses where the columns are the chords and the beams and braces are the web members. Braced frames may act alone or in conjunction with concrete or masonry walls, or steel moment frames, to form a dual system.

Steel braced frames may be divided into two types: concentric braced frames (CBF) and eccentric braced frames (EBF). Columns, beams, braces, and connections are the components of CBF and EBF. A link beam is also a component of an EBF. The components are usually hot-rolled shapes. The components may be bare steel, steel with a nonstructural coating for fire protection, steel with concrete encasement for fire protection, or steel with masonry encasement for fire protection.

5.5.2 Concentric Braced Frames (CBF)

5.5.2.1 General

Concentric braced frames are braced systems whose worklines essentially intersect at points. Minor eccentricities, where the worklines intersect within the width of the bracing member are acceptable if accounted for in the design.

5.5.2.2 Stiffness for Analysis

A. Linear Static and Dynamic Procedures

Beams and Columns. Axial area, shear area, and moment of inertia shall be calculated as given in Section 5.4.2.2.

Connections. FR connections shall be modeled as given in Section 5.4.2.2. PR connections shall be modeled as given in Section 5.4.3.2.

Braces. Braces shall be modeled the same as columns for linear procedures.

B. Nonlinear Static Procedure

- Use elastic component properties as given in Section 5.4.2.2A.
- Use appropriate nonlinear moment curvature or load-deformation behavior for beams, columns, braces, and connections to represent yielding and buckling. Guidelines are given in Section 5.4.2.2 for beams and columns and Section 5.4.3.2 for PR connections.

Braces. Use nonlinear load-deformation behavior for braces as determined by experiment or analysis supported by experiment. In lieu of these, the load versus axial deformation relationship given in Figure 5-1 and Table 5-8 may be used. The parameters Δ and Δ_y are axial deformation and axial deformation at brace buckling. The reduction in strength of a brace after buckling must be included in the model. Elasto-plastic brace behavior may be assumed for the compression brace if the yield force is taken as the residual strength after buckling, as indicated by the parameter *c* in Figure 5-1 and Table 5-8. Implications of forces higher than this lower-bound force must be considered.

C. Nonlinear Dynamic Procedure

The complete hysteretic behavior of each component must be properly based on experiment or generally accepted engineering practice. Guidelines for this are given in the *Commentary.*

5.5.2.3 Strength and Deformation Acceptance Criteria

A. Linear Static and Dynamic Procedures

The strength and deformation acceptance criteria for these methods require that the load and resistance relationships given in Equations 3-18 and 3-19 in Chapter 3 be satisfied. The design strength and other restrictions for a beam and column shall be determined in accordance with the provisions given in Section 5.4.2.3.

Evaluation of component acceptability requires knowledge of the component lower-bound capacity, Q_{CL}, for Equation 3-19 and Q_{CE} for Equation 3-18, and the ductility factor, *m*, as given in Table 5-7 for use in Equation 3-10. Columns shall be considered to be force-controlled members. Values for Q_{CE} and Q_{CL} for beams and columns are the same as those given in Section 5.4.2.3 for FR frames. Q_{CE} and Q_{CL} for PR connections are given in Section 5.4.3.3B. Braces are deformation-controlled components where the expected strength for the brace in compression is computed in the same manner as for columns given in Section 5.4.2.3.

Table 5-7 Acceptance Criteria for Linear Procedures—Braced Frames and Steel Shear Walls

Component/Action	*m* Values for Linear Procedures				
	Primary			Secondary	
	IO	LS	CP	LS	CP
Concentric Braced Frames					
Columns:[1]					
a. Columns in compression[1]	Force-controlled member, use Equation 3-15 or 3-16.				
b. Columns in tension[1]	1	3	5	6	7
Braces in Compression[2]					
a. Double angles buckling in plane	0.8	6	8	7	9
b. Double angles buckling out of plane	0.8	5	7	6	8
c. W or I shape	0.8	6	8	6	8
d. Double channel buckling in plane	0.8	6	8	7	9
e. Double channel buckling out of plane	0.8	5	7	6	8
f. Rectangular concrete-filled cold-formed tubes	0.8	5	7	5	7

Table 5-7 Acceptance Criteria for Linear Procedures—Braced Frames and Steel Shear Walls (continued)

Component/Action	*m* Values for Linear Procedures				
	Primary			Secondary	
	IO	LS	CP	LS	CP
g. Rectangular cold-formed tubes					
1. $\frac{d}{t} \le \frac{90}{\sqrt{F_y}}$	0.8	5	7	5	7
2. $\frac{d}{t} \ge \frac{190}{\sqrt{F_y}}$	0.8	2	3	2	3
3. $\frac{90}{\sqrt{F_y}} \le \frac{d}{t} \le \frac{190}{\sqrt{F_y}}$	Use linear interpolation				
h. Circular hollow tubes					
1. $\frac{d}{t} \le \frac{1500}{F_y}$	0.8	5	7	5	7
2. $\frac{d}{t} \ge \frac{6000}{F_y}$	0.8	2	3	2	3
3. $\frac{1500}{F_y} \le \frac{d}{t} \le \frac{6000}{F_y}$	Use linear interpolation				
Braces in Tension[3]	1	6	8	8	10
Eccentric Braced Frames					
a. Beams	Governed by link				
b. Braces	Force-controlled, use Equation 3-19				
c. Columns in compression	Force-controlled, use Equation 3-19				
d. Columns in tension	1	3	5	6	7
Link beam[4]					
a.[5] $\frac{2M_{CE}}{eV_{CE}} < 1.6$ d: 16, e: 18, c: 1.00	1.5	9	13	13	15
b. $\frac{2M_{CE}}{eV_{CE}} < 2.6$	Same as for beam in FR moment frame; see Table 5-3				
c. $1.6 \le \frac{2M_{CE}}{eV_{CE}} < 2.6$	Use linear interpolation				

Table 5-7 Acceptance Criteria for Linear Procedures—Braced Frames and Steel Shear Walls (continued)

	m Values for Linear Procedures				
	Primary			Secondary	
Component/Action	**IO**	**LS**	**CP**	**LS**	**CP**
Steel Shear Walls[6]	1.5	8	12	12	14

1. Columns in moment or braced frames need only be designed for the maximum force that can be delivered.
2. Connections in braced frames should be able to carry 1.25 times the brace strength in compression, or the expected strength of the member in tension. Otherwise maximum value of $m = 2$.
3. For tension-only bracing systems, divide these *m* values by 2.
4. Assumes ductile detailing for flexural links.
5. Link beams with three or more web stiffeners. If no stiffeners, use half of these values. For one or two stiffeners, interpolate.
6. Applicable if stiffeners are provided to prevent shear buckling.

For common cross bracing configurations where both braces are attached to a common gusset plate where they cross at their midpoints, the effective length of each brace may be taken as 0.5 times the total length of the brace including gusset plates for both axes of buckling. For other bracing configurations (chevron, V, single brace), if the braces are back-to-back shapes attached to common gusset plates, the length shall be taken as the total length of the brace including gusset plates, and *K*, the effective length factor, (AISC, 1994a) may be assumed to be 0.8 for in-plane buckling and 1.0 for out-of-plane buckling.

Restrictions on bracing members, gusset plates, brace configuration, and lateral bracing of link beams are given in the seismic provisions of AISC (1994a). If the special requirements of Section 22.11.9.2 of AISI (1986) are met, then 1.0 may be added to the brace *m* values given in Table 5-7.

The strength of brace connections shall be the larger of the maximum force deliverable by the tension brace or 1.25 times the maximum force deliverable by the compression brace. If not, the connection shall be strengthened, or the *m* values and deformation limits shall be reduced to comparable values given for connectors with similar limit states (see Table 5-5).

Stitch plates for built-up members shall be spaced such that the largest slenderness ratio of the components of the brace is at most 0.4 times the governing slenderness ratio of the brace as a whole. The stitches for compressed members must be able to resist 0.5 times the maximum brace force where buckling of the brace will cause shear forces in the stitches. If not, stitch plates shall be added, or the *m* values in Table 5-7 and deformation limits in Table 5-8 shall be reduced by 50%. Values of *m* need not be less than 1.0.

B. Nonlinear Static Procedure

The NSP requires modeling of the complete nonlinear force-deformation relationship to failure for each component. This may be based on experiment, or analysis verified by experiment. Guidelines are given in the *Commentary*. In lieu of these, the conservative approximate behavior depicted in Figure 5-1 may be used. The values for Q_{CE} and θ_y are the same as those used for the LSP. Deformation parameters *c*, *d*, and *e* for Figure 5-1 and deformation limits are given in Table 5-8. The force-deformation relationship for the compression brace should be modeled as accurately as possible (see the *Commentary*). In lieu of this, the brace may be assumed to be elasto-plastic, with the yield force equal to the residual force that corresponds to the parameter *c* in Figure 5-1 and Table 5-8. This assumption is an estimate of the lower-bound brace force. Implications of forces higher than this must be considered.

C. Nonlinear Dynamic Procedure

The complete hysteretic behavior of each component must be modeled for this procedure. Guidelines for this are given in the *Commentary*.

Table 5-8 Modeling Parameters and Acceptance Criteria for Nonlinear Procedures—Braced Frames and Steel Shear Walls

Component/Action	$\frac{\Delta}{\Delta_y}$ d	$\frac{\Delta}{\Delta_y}$ e	Residual Force Ratio c	Deformation Primary IO	Deformation Primary LS	Deformation Primary CP	Deformation Secondary LS	Deformation Secondary CP
Concentric Braced Frames								
a. Columns in compression[1]	Force-controlled, use Equation 3-19							
b. Columns in tension[1]	6	8	1.000	1	4	6	7	8
Braces in Compression[2,3]								
a. Two angles buckle in plane	1	10	0.2	0.8	6	8	8	9
b. Two angles buckle out of plane	1	9	0.2	0.8	5	7	7	8
c. W or I shape	1	9	0.2	0.8	6	8	8	9
d. Two channels buckle in plane	1	10	0.2	0.8	6	8	8	9
e. Two channels buckle out of plane	1	9	0.2	0.8	5	7	7	8
f. Concrete-filled tubes	1	8	0.2	0.8	5	7	7	8
g. Rectangular cold-formed tubes								
1. $\frac{d}{t} \le \frac{90}{\sqrt{F_y}}$	1	8	0.4	0.8	5	7	7	8
2. $\frac{d}{t} \ge \frac{190}{\sqrt{F_y}}$	1	4	0.2	0.8	2	3	3	4
3. $\frac{90}{\sqrt{F_y}} \le \frac{d}{t} \le \frac{190}{\sqrt{F_y}}$	Use linear interpolation							
h. Circular hollow tubes								
1. $\frac{d}{t} \le \frac{1500}{F_y}$	1	10	0.4	0.8	5	7	6	9
2. $\frac{d}{t} \ge \frac{6000}{F_y}$	1	4	0.2	0.8	2	3	3	4
3. $\frac{1500}{F_y} \le \frac{d}{t} \le \frac{6000}{F_y}$	Use linear interpolation							
Braces in Tension	12	15	0.800	1	8	10	12	14
Eccentric Braced Frames								
a. Beams	Governed by link							
b. Braces	Force-controlled, use Equation 3-19							
c. Columns in compression	Force-controlled, use Equation 3-19							
d. Columns in tension	6	8	1.000	1	4	6	7	8

Table 5-8 Modeling Parameters and Acceptance Criteria for Nonlinear Procedures—Braced Frames and Steel Shear Walls (continued)

	$\frac{\Delta}{\Delta_y}$		Residual Force Ratio	Deformation				
				Primary			Secondary	
Component/Action	***d***	***e***	***c***	**IO**	**LS**	**CP**	**LS**	**CP**
Link Beam[3]								
a.[4] $\frac{2M_{CE}}{eV_{CE}} \leq 1.6$	16	18	0.80	1.5	12	15	15	17
b. $\frac{2M_{CE}}{eV_{CE}} \geq 2.6$	Same as for beam in FR moment frame (see Table 5-4)							
c. $1.6 \leq \frac{2M_{CE}}{eV_{CE}} \leq 2.6$	Use linear interpolation							
Steel Shear Walls[5]	15	17	.07	1.5	11	14	14	16

1. Columns in moment or braced frames need only be designed for the maximum force that can be delivered.
2. Δ_c is the axial deformation at expected buckling load.
3. Deformation is rotation angle between link and beam outside link or column. Assume Δ_y is 0.01 radians for short links.
4. Link beams with three or more web stiffeners. If no stiffeners, use half of these values. For one or two stiffeners, interpolate.
5. Applicable if stiffeners are provided to prevent shear buckling.

5.5.2.4 Rehabilitation Measures for Concentric Braced Frames

Provisions for moment frames may be applicable to braced frames. Braces that are insufficient in strength and/or ductility may be replaced or modified. Insufficient connections may also be modified. Columns may be encased in concrete to improve their performance. For further guidance, see Section 5.4.2.4 and the *Commentary.*

5.5.3 Eccentric Braced Frames (EBF)

5.5.3.1 General

For an EBF, the action lines of the braces do not intersect at the action line of the beam. The distance between the brace action lines where they intersect the beam action line is the eccentricity, *e*. The beam segment between these points is the link beam. The strength of the frame is governed by the strength of the link beam.

5.5.3.2 Stiffness for Analysis

A. Linear Static and Dynamic Procedures

The elastic stiffness of beams, columns, braces, and connections are the same as those used for FR and PR moment frames and CBF. The load-deformation model for a link beam must include shear deformation and flexural deformation.

The elastic stiffness of the link beam, K_e, is

$$K_e = \frac{K_s K_b}{K_s + K_b} \tag{5-26}$$

where

$$K_s = \frac{GA_w}{e} \tag{5-27}$$

and

$$K_b = \frac{12EI_b}{e^3} \tag{5-28}$$

where

A_w = $(d_b - 2t_f)\ t_w$, in.^{2}
e = Length of link beam, in.
G = Shear modulus, k/in.^{2}
K_e = Stiffness of the link beam, k/in.
K_b = Flexural stiffness, kip/in.
K_s = Shear stiffness, kip/in.

The strength of the link beam may be governed by shear, flexure, or the combination of these.

If $e \le \frac{1.6M_{CE}}{V_{CE}}$

$$Q_{CE} = V_{CE} = 0.6F_{ye}t_wA_w \tag{5-29}$$

where

M_{CE} = Expected moment, kip/in.

If $e > \frac{2.6M_{CE}}{V_{CE}}$

$$Q_{CE} = V_{CE} = 2\frac{M_{CE}}{e} \tag{5-30}$$

If $\frac{1.6M_{CE}}{V_{CE}} \le e \le \frac{2.6M_{CE}}{V_{CE}}$, use linear interpolation between Equations 5-29 and 5-30.

The yield deformation is the link rotation as given by

$$\theta_y = \frac{Q_{CE}}{K_e e} \tag{5-31}$$

B. Nonlinear Static Procedure

The NSP requires modeling of the complete nonlinear load-deformation relation to failure for each component. This may be based on experiment, or rational analysis verified by experiment. In lieu of these, the load versus deformation relationship given in Figure 5-1 and Table 5-8 may be used. Q_{CE} and θ_y are calculated in accordance with provisions given in AISC (1994a) or by rational analysis.

The nonlinear models used for beams, columns, and connections for FR and PR moment frames, and for the braces for a CBF, may be used.

C. Nonlinear Dynamic Procedure

The strength and deformation criteria require that the load and resistance relationships given in Equations 3-18 and 3-19 in Chapter 3 be satisfied. The complete hysteretic behavior of each component must be modeled for this procedure. Guidelines for this are given in the *Commentary.*

5.5.3.3 Strength and Deformation Acceptance Criteria

A. Linear Static and Dynamic Procedures

The modeling assumptions given for a CBF are the same as those for an EBF. Values for Q_{CE} and θ_y are given in Section 5.5.3.2 and m values are given in Table 5-7. The strength and deformation capacities of the link beam may be governed by shear strength, flexural strength, or their interaction. The values of Q_{CE} and θ_y are the same as those used in the LSP as given in Section 5.5.3.2A. Links and beams are deformation-controlled components and must satisfy Equation 3-18. Columns and braces are to be considered force-controlled members and must satisfy Equation 3-19.

The requirements for link stiffeners, link-to-column connections, lateral supports of the link, the diagonal brace and beam outside the link, and beam-to-column connections given in AISC (1994a) must be met. The brace should be able to carry 1.25 times the link strength to ensure link yielding without brace or column buckling. If this is not satisfied for existing buildings, the design professional shall make extra efforts to verify that the expected link strength will be reached before brace or column buckling. This may require additional inspection and material testing. Where the link beam is attached to the column flange with full-pen welds, the provisions for these connections is the same as for FR frame full-pen connections. The columns of an EBF are force-controlled members. The maximum force deliverable to a column should be calculated from the maximum brace forces equal to 1.25 times the calculated strength of the brace.

B. Nonlinear Static Procedure

The NSP requirements for an EBF are the same as those for a CBF. Modeling of the nonlinear load deformation of the link beam should be based on experiment, or rational analysis verified by experiment. In lieu of these, the conservative approximate behavior depicted in Figure 5-1 may be used. Values for Q_{CE} and θ_y are the same as those used for the LSP. Deformation limits are given in Table 5-8.

C. Nonlinear Dynamic Procedure

The complete hysteretic behavior of each component must be properly modeled. This behavior must be verified by experiment. This procedure is not recommended in most cases.

5.5.3.4 Rehabilitation Measures for Eccentric Braced Frames

Many of the beams, columns, and braces may be rehabilitated using procedures given for moment frames and CBFs. Cover plates and/or stiffeners may be used for these components. The strength of the link beam may be increased by adding cover plates to the beam flange(s), adding doubler plates or stiffeners to the web, or changing the brace configuration.

5.6 Steel Plate Walls

5.6.1 General

A steel plate wall develops its seismic resistance through shear stress in the plate wall. In essence, it is a steel shear wall. A solid steel plate, with or preferably without perforations, fills an entire bay between columns and beams. The steel plate is welded to the columns on each side and to the beams above and below. Although these are not common, they have been used to rehabilitate a few essential structures where immediate occupancy and operation of a facility is mandatory after a large earthquake. These walls work in conjunction with other existing elements to resist seismic load. However, due to their stiffness, they attract much of the seismic shear. It is essential that the new load paths be carefully established.

5.6.2 Stiffness for Analysis

5.6.2.1 Linear Static and Dynamic Procedures

The most appropriate way to analyze a steel plate wall is to use a plane stress finite element model with the beams and columns as boundary elements. The global stiffness of the wall can be calculated. The modeling can be similar to that used for a reinforced concrete shear wall. A simple approximate stiffness K_w for the wall is

$$K_w = \frac{Ga\ t_w}{h} \qquad (5\text{-}32)$$

where

G = Shear modulus of steel, ksi
a = Clear width of wall between columns, in.
h = Clear height of wall between beams, in.
t_w = Thickness of plate wall, in.

Other approximations of the wall stiffness based on principles of mechanics are acceptable.

5.6.2.2 Nonlinear Static Procedure

The elastic part of the load-deformation relationship for the wall is given in Section 5.6.2.1. The yield load, Q_{CE}, is given in the next section. The complete nonlinear load-deformation relationship should be based on experiment or rational analysis. In lieu of this, the approximate simplified behavior may be modeled using Figure 5-1 and Table 5-8.

5.6.2.3 Nonlinear Dynamic Procedure

The complete hysteretic behavior of each component must be properly modeled. This behavior must be verified by experiment. This procedure is not recommended in most cases.

5.6.3 Strength and Deformation Acceptance Criteria

5.6.3.1 Linear Static and Dynamic Procedures

The strength and deformation acceptance criteria for these methods require that the load and resistance relationships given in Equations 3-18 and 3-19 in Chapter 3 be satisfied. The design strength of the steel wall shall be determined using the appropriate equations in Part 6 of AISC (1994a). The wall can be assumed to be like the web of a plate girder. Design restrictions for plate girder webs given in AISC (1994a), particularly those related to stiffener spacing, must be followed. Stiffeners should be spaced such that buckling of the wall does not occur. In this case

$$Q_{CE} = V_{CE} = 0.6F_{ye}at_w \tag{5-33}$$

In lieu of stiffeners, the steel wall may be encased in concrete. If buckling is not prevented, equations for V_{CE} given in AISC (1994a) for plate girders may be used. The *m* values for steel walls are given in Table 5-7. A steel shear wall is a deformation-controlled component.

5.6.3.2 Nonlinear Static Procedure

The NSP requires modeling of the complete load-deformation behavior to failure. This may be based on experiment or rational analysis. In lieu of these, the conservative approximate behavior depicted in Figure 5-1 may be used, along with parameters given in Table 5-8. The equation for Q_{CE} is Equation 5-33. The yield deformation is

$$\theta_y = \frac{Q_{CE}}{K_w} \tag{5-34}$$

5.6.3.3 Nonlinear Dynamic Procedure

The complete hysteretic behavior of each component must be properly modeled. This behavior must be verified by experiment. This procedure is not recommended in most cases.

5.6.4 Rehabilitation Measures

This is not an issue because steel walls in existing construction are rare.

5.7 Steel Frames with Infills

It is common for older existing steel frame buildings to have complete or partial infill walls of reinforced concrete or masonry. Due to the high wall stiffness relative to the frame stiffness, the infill walls will attract most of the seismic shear. In many cases, because these walls are unreinforced or lightly reinforced, their strength and ductility may be inadequate.

The engineering properties and acceptance criteria for the infill walls are presented in Chapter 6 for concrete and Chapter 7 for masonry. The walls may be considered to carry all of the seismic shear in these elements until complete failure of the walls has occurred. After that, the steel frames will resist the seismic forces. Before the loss of the wall, the steel frame adds confining pressure to the wall and enhances its resistance. However, the actual effective forces on the steel frame components are probably minimal. As the frame components begin to develop force they will deform; however, the concrete or masonry on the other side is stiffer so it picks up the load.

The analysis of the component should be done in stages and carried through each performance goal. At the point where the infill has been deemed to fail—as given in Chapter 6 or Chapter 7—the wall should be removed from the analytical model and the analysis resumed with only the bare steel frame in place. At this point, the engineering properties and acceptance criteria for the moment frame given above in Section 5.4 are applicable.

5.8 Diaphragms

5.8.1 Bare Metal Deck Diaphragms

5.8.1.1 General

Bare metal deck diaphragms are usually used for roofs of buildings where there are very light gravity loads other than support of roofing materials. The metal deck units are often composed of gage thickness steel sheets, from 22 gage down to 14 gage, two to three feet wide, and formed in a repeating pattern with ridges and valleys. Rib depths vary from 1-1/2 to 3 inches in most cases. Decking units are attached to each other and to the structural steel supports by welds or, in some more recent applications, by mechanical fasteners. In large roof structures, these roofs may have supplementary diagonal bracing. (See the description of horizontal steel bracing in Section 5.8.4.)

Chord and collector elements in these diaphragms are considered to be composed of the steel frame elements attached to the diaphragm. Load transfer to frame elements that act as chords or collectors in modern frames is through shear connectors, puddle welds, screws, or shot pins.

5.8.1.2 Stiffness for Analysis

A. Linear Static Procedure

The distribution of forces for existing diaphragms is based on flexible diaphragm assumption, with diaphragms acting as simply supported between the stiff vertical lateral-force-resisting elements. Flexibility factors for various types of metal decks are available from manufacturers' catalogs. In systems for which values are not available, values can be established by interpolating between the most representative systems for which values are available. Flexibility can also be calculated using the Steel Deck Institute Diaphragm Design Manual (Section 3). The analysis should verify that the diaphragm strength is not exceeded for the elastic assumption to hold.

All criteria for existing diaphragms mentioned above apply to stiffened or strengthened diaphragms. Interaction of new and existing elements of strengthened diaphragms must be considered to ensure stiffness compatibility. Load transfer mechanisms between new and existing diaphragm elements must be considered.

Analyses should verify that diaphragm strength is not exceeded, so that elastic assumptions are still valid.

B. Nonlinear Static Procedure

Inelastic properties of diaphragms are usually not included in inelastic seismic analyses.

More flexible diaphragms, such as bare metal deck or deck-formed slabs with long spans between lateral-force-resisting elements, could be subject to inelastic action. Procedures for developing models for inelastic response of wood diaphragms in unreinforced masonry (URM) buildings could be used as the basis for an inelastic model of a bare metal deck diaphragm condition. A strain-hardening modulus of 3% could be used in the post-elastic region. If the weak link of the diaphragm is connection failure, then the element nonlinearity cannot be incorporated into the model.

5.8.1.3 Strength and Deformation Acceptance Criteria

Member capacities of steel deck diaphragms are given in International Conference of Building Officials (ICBO) reports, in manufacturers' literature, or in the publications of the Steel Deck Institute (SDI). (See the references in Section 5.12 and *Commentary* Section C5.12.) Where allowable stresses are given, these may be multiplied by 2.0 in lieu of information provided by the manufacturer or other knowledgeable sources. If bare deck capacity is controlled by connections to frame members or panel buckling, then inelastic action and ductility are limited. Therefore, the deck should be considered to be a force-controlled member.

In many cases, diaphragm failure would not be a life safety consideration unless it led to a loss of bearing support or anchorage. Goals for higher performance would limit the amount of damage to the connections to insure that the load transfer mechanism was still intact. Deformations should be limited to below the threshold of deflections that cause damage to other elements (either structural or nonstructural) at specified Performance Levels.

The m value for shear yielding, or panel or plate buckling is 1, 2, or 3 for the IO, LS, or CP Performance Levels, respectively. Weld and connector failure is force-controlled.

The SDI calculations procedure should be used for strengths, or ICBO values with a multiplier may be used to bring allowable values to expected strength levels. Specific references are given in Section 5.12 and in the *Commentary,* Section C5.12.

Connections between metal decks and steel framing commonly use puddle welds. Connection capacity must be checked for the ability to transfer the total diaphragm reaction into the steel framing. Connection capacities are provided in ICBO reports, manufacturers' data, the SDI Manual, or the Welding Code for Sheet Steel, AWS D1.3. Other attachment systems, such as clips, are sometimes used.

5.8.1.4 Rehabilitation Measures

See the *Commentary.*

5.8.2 Metal Deck Diaphragms with Structural Concrete Topping

5.8.2.1 General

Metal deck diaphragms with structural concrete topping are frequently used on floors and roofs of buildings where there are typical floor gravity loads. The metal deck may be either a composite deck, which has indentations, or a noncomposite form deck. In both types of deck, the slab and deck act together to resist diaphragm loads. The concrete fill may be either normal or lightweight concrete, with reinforcing composed of wire mesh or small-diameter reinforcing steel. Additional slab reinforcing may be added at areas of high stress. The metal deck units are composed of gage thickness steel sheets, two to three feet wide, and are formed in a repeating pattern with ridges and valleys. Decking units are attached to each other and to structural steel supports by welds or, in some more recent applications, by mechanical fasteners. Concrete diaphragms in which the slab was formed and the beams are encased in concrete for fire protection may be considered to be similar to topped metal deck diaphragms.

Concrete has structural properties that significantly add to diaphragm stiffness and strength. Concrete reinforcing ranges from light mesh reinforcement to a regular grid of small reinforcing bars (#3 or #4). Metal decking is typically composed of corrugated sheet steel from 22 ga. down to 14 ga. Rib depths vary from 1-1/2 to 3 inches in most cases. Attachment of the metal deck to the steel frame is usually accomplished using puddle welds at one to two feet on center. For composite behavior, shear studs are welded to the frame before the concrete is cast.

Chord and collector elements in these diaphragms are considered to be composed of the steel frame elements attached to the diaphragm. Load transfer to frame elements that act as chords or collectors in modern frames is usually through puddle welds or headed studs. In older construction where the frame is encased for fire protection, load transfer is made through bond.

5.8.2.2 Stiffness for Analysis

A. Linear Static Procedure

For existing diaphragms, the distribution of forces may be based on a rigid diaphragm assumption if the diaphragm span-to-depth ratio is not greater than five to one. For greater ratios, justify with analysis. Diaphragm flexibility should be included in cases with larger spans and/or plan irregularities by three-dimensional analysis procedures and shell finite elements for the diaphragms. Diaphragm stiffness can be calculated using the SDI Design Manual, manufacturers' catalogs, or with a representative concrete thickness.

All procedures for existing diaphragms noted above apply to strengthened diaphragms as well. Interaction of new and existing elements of strengthened diaphragms (stiffness compatibility) must be considered. Load transfer mechanisms between new and existing diaphragm components may need to be considered in determining the flexibility of the diaphragm.

All procedures for existing diaphragms noted above apply to new diaphragms. Interaction of new diaphragms with the existing frames must be considered. Load transfer mechanisms between new diaphragm components and existing frames may need to be considered in determining the flexibility of the diaphragm.

For all diaphragms, the analyses must verify that the diaphragm strength is not exceeded, so that elastic assumptions are still valid.

B. Nonlinear Static Procedure

Inelastic properties of diaphragms are usually not included in inelastic seismic analyses, but could be if the connections are adequate. More flexible diaphragms—such as bare metal deck or deck-formed slabs with long spans between lateral-force-resisting elements—could be subject to inelastic action. Procedures for developing models for inelastic response of wood diaphragms in URM buildings could be used as the basis for an inelastic model of a bare metal deck or long span composite diaphragm condition. If the weak link of the diaphragm is connection failure, the element nonlinearity cannot be incorporated into the model.

5.8.2.3 Strength and Deformation Acceptance Criteria

Member capacities of steel deck diaphragms with structural concrete are given in manufacturers' catalogs, ICBO reports, or the SDI Manual. If composite deck capacity is controlled by shear connectors, inelastic action and ductility are limited. It would be expected that there would be little or no inelastic action in steel deck/concrete diaphragms, except in long span conditions; however, perimeter transfer mechanisms and collector forces must be considered to be sure that this is the case.

In many cases, diaphragm failure would not be a life safety consideration unless it led to a loss of bearing support or anchorage. Goals for higher performance would limit the amount of damage to the connections or cracking in concrete-filled slabs in order to ensure that the load transfer mechanism was still intact. Deformations should be limited below the threshold of deflections that cause damage to other elements (either structural or nonstructural) at specified Performance Levels.

Connection failure is force-limited, so Equation 3-19 must be used. Shear failure of the deck requires cracking of the concrete and/or tearing of the metal deck, so the *m* values for IO, LS, and CP Performance Levels are 1, 2, and 3, respectively. See Section 5.8.6.3 for acceptance criteria for collectors.

SDI calculation procedures should be used for strengths, or ICBO values with a multiplier of 2.0 should be used to bring allowable values to a strength level. The deck will be considered elastic in most analyses.

Connector capacity must be checked for the ability to transfer the total diaphragm reaction into the supporting steel framing. This load transfer can be achieved by puddle welds and/or headed studs. For the connection of the metal deck to steel framing, puddle welds to beams are most common. Connector capacities are provided in ICBO reports, manufacturers' data, the SDI Manual, or the Welding Code for Sheet Steel, AWS D1.3. Shear studs replace puddle welds to beams where they are required for composite action with supporting steel beams.

Headed studs are most commonly used for connection of the concrete slab to steel framing. Connector capacities can be found using the AISC Manual of Steel Construction, UBC, or manufacturers' catalogs. When steel beams are designed to act compositely with the slab, shear connectors must have the capacity to transfer both diaphragm shears and composite beam shears. In older structures where the beams are encased in concrete, load transfer may be provided through bond between the steel and concrete.

5.8.2.4 Rehabilitation Measures

See the *Commentary.*

5.8.3 Metal Deck Diaphragms with Nonstructural Concrete Topping

5.8.3.1 General

Metal deck diaphragms with nonstructural concrete fill are typically used on roofs of buildings where there are very small gravity loads. The concrete fill, such as very lightweight insulating concrete (e.g., vermiculite), does not have usable structural properties. If the concrete is reinforced, reinforcing consists of wire mesh or small-diameter reinforcing steel. Typically, the metal deck is a form deck or roof decks, so the only attachment between the concrete and metal deck is through bond and friction. The concrete fill is not designed to act compositely with the metal deck and has no positive structural attachment. The metal deck units are typically composed of gage thickness steel sheets, two to three feet wide, and formed in a repeating pattern with ridges and valleys. Decking units are attached to each other and structural steel supports by welds or, in some more recent applications, by mechanical fasteners.

Consideration of any composite action must be done with caution, after extensive investigation of field conditions. Material properties, force transfer mechanisms, and other similar factors must be verified in order to include such composite action. Typically, the decks are composed of corrugated sheet steel from 22 gage down to 14 gage, and the rib depths vary from 9/16 to 3 inches in most cases. Attachment to the steel frame is usually through puddle welds, typically spaced at one to two feet on center. Chord and collector elements in these diaphragms are composed of the steel frame elements attached to the diaphragm.

5.8.3.2 Stiffness for Analysis

A. Linear Static Procedure

The potential for composite action and modification of load distribution must be considered. Flexibility of the diaphragm will depend on the strength and thickness of the topping. It may be necessary to bound the solution in some cases, using both rigid and flexible diaphragm assumptions. Interaction of new and existing elements of strengthened diaphragms (stiffness compatibility) must be considered, and the load transfer mechanisms between the new and existing diaphragm elements may need to be considered in determining the flexibility of the diaphragm. Similarly, the interaction of new diaphragms with existing frames must be carefully considered, as well as the load transfer mechanisms between them. Finally, the analyses must verify that diaphragm strength is not exceeded, so elastic assumptions are still valid.

B. Nonlinear Static Procedure

Inelastic properties of diaphragms are usually not included in inelastic seismic analyses. When nonstructural topping is present its capacity must be verified. More flexible diaphragms, such as bare metal deck or decks with inadequate nonstructural topping, could be subject to inelastic action. Procedures for developing models for inelastic response of wood diaphragms in URM buildings could be used as the basis for an inelastic model of a bare metal deck diaphragm condition. If a weak link of the diaphragm is connection failure, then the element nonlinearity cannot be incorporated into the model.

5.8.3.3 Strength and Deformation Acceptance Criteria

A. Linear Static Procedure

Capacities of steel deck diaphragms with nonstructural topping are provided by ICBO reports, by manufacturers, or in general by the SDI Manual. When the connection failure governs, or topping lacks adequate strength, inelastic action and ductility are limited. As a limiting case, the diaphragm shear may be computed using only the bare deck (see Section 5.8.1 for bare decks). Generally, there should be little or no inelastic action in the diaphragms, provided the connections to the framing members are adequate.

In many cases, diaphragm failure would not be a life safety consideration unless it led to a loss of bearing support or anchorage. Goals for higher Performance Levels would limit the amount of damage to the connections or cracking in concrete filled slabs, to ensure that the load transfer mechanism was still intact. Deformations should be limited below the threshold of deflections that cause damage to other elements (either structural or nonstructural) at specified performance levels.

Connection failure is force-limited, so Equation 3-19 must be used. Shear failure of the deck requires concrete cracking and/or tearing of the metal deck, so m values for IO, LS, and CP are 1, 2, and 3, respectively. Panel buckling or plate buckling have m values of 1, 2, and 3 for IO, LS, and CP. SDI calculation procedures should be used for strengths, or ICBO values with a multiplier to bring allowable values to strength levels.

5.8.3.4 Rehabilitation Measures

See the *Commentary.*

5.8.4 Horizontal Steel Bracing (Steel Truss Diaphragms)

5.8.4.1 General

Horizontal steel bracing (steel truss diaphragms) may be used in conjunction with bare metal deck roofs and in conditions where diaphragm stiffness and/or strength is inadequate to transfer shear forces. Steel truss diaphragm elements are typically found in conjunction with vertical framing systems that are of structural steel framing. Steel trusses are more common in long span situations, such as special roof structures for arenas, exposition halls, auditoriums, and industrial buildings. Diaphragms with a large span-to-depth ratio may often be stiffened by the addition of steel trusses. The addition of steel trusses for diaphragms identified to be deficient may provide a proper method of enhancement.

Horizontal steel bracing (steel truss diaphragms) may be made up of any of the various structural shapes. Often, the truss chord elements consist of wide flange shapes that also function as floor beams to support the gravity loads of the floor. For lightly loaded conditions, such as industrial metal deck roofs without concrete fill, the diagonal members may consist of threaded rod elements, which are assumed to act only in tension. For steel truss diaphragms with large loads, diagonal elements may consist of wide flange members, tubes, or other structural elements that will act in both tension and compression. Truss element connections are generally concentric, to provide the maximum lateral stiffness and ensure that the truss members act under pure axial load. These connections are generally similar to those of gravity-load-resisting trusses. Where concrete fill is provided over the metal decking, consideration of relative rigidities between the truss and concrete systems may be necessary.

5.8.4.2 Stiffness for Analysis

A. Linear Static Procedure

Existing truss diaphragm systems are modeled as horizontal truss elements (similar to braced steel frames) where axial stiffness controls the deflections. Joints are often taken as pinned. Where joints provide the ability for moment resistance or where eccentricities are introduced at the connections, joint rigidities should be considered. A combination of stiffness with that of concrete fill over metal decking may be necessary in some instances. Flexibility of truss diaphragms should be considered in distribution of lateral loads to vertical elements.

The procedures for existing diaphragms provided above apply to strengthened truss diaphragms. Interaction of new and existing elements of strengthened diaphragm systems (stiffness compatibility) must be considered in cases where steel trusses are added as part of a seismic upgrade. Load transfer mechanisms between new and existing diaphragm elements must be considered in determining the flexibility of the strengthened diaphragm.

The procedures for existing truss diaphragms mentioned above also apply to new diaphragms. Interaction of new truss diaphragms with existing frames must be considered. Load transfer mechanisms between new diaphragm elements and existing frames may need to be considered in determining the flexibility of the diaphragm/frame system.

For modeling assumptions and limitations, see the preceding comments related to truss joint modeling, force transfer, and interaction between diaphragm elements. Analyses are also needed to verify that elastic diaphragm response assumptions are still valid.

Acceptance criteria for the components of a truss diaphragm are the same as for a CBF.

B. Nonlinear Static Procedure

Inelastic properties of truss diaphragms are usually not included in inelastic seismic analyses. In the case of truss diaphragms, inelastic models similar to those of braced steel frames may be appropriate. Inelastic deformation limits of truss diaphragms may be different from those prescribed for braced steel frames (e.g., more consistent with that of a concrete-topped diaphragm).

5.8.4.3 Strength and Deformation Acceptance Criteria

Member capacities of truss diaphragm members may be calculated in a manner similar to those for braced steel frame members. It may be necessary to include gravity force effects in the calculations for some members of these trusses. Lateral support conditions provided by metal deck, with or without concrete fill, must be properly considered. Force transfer mechanisms between various members of the truss at the connections, and between trusses and frame elements, must be considered to verify the completion of the load path.

In many cases, diaphragm distress would not be a life safety consideration unless it led to a loss of bearing support or anchorage. Goals for higher Performance Levels would limit the amount of damage to the connections or bracing elements, to insure that the load transfer mechanism was still complete. Deformations should be limited below the threshold of deflections that cause damage to other elements (either structural or nonstructural) at specified Performance Levels. These values must be established in conjunction with those of braced steel frames.

The *m* values to be used are half of those for components of a CBF as given in Table 5-7.

A. Nonlinear Static Procedure

Procedures similar to those used for a CBF should be used, but deformation limits shall be half of those given for CBFs in Table 5-8.

5.8.4.4 Rehabilitation Measures

See the *Commentary.*

5.8.5 Archaic Diaphragms

5.8.5.1 General

Archaic diaphragms in steel buildings generally consist of shallow brick arches that span between steel floor beams, with the arches packed tightly between the beams to provide the necessary resistance to thrust forces. Archaic steel diaphragm elements are almost always found in older steel buildings in conjunction with vertical systems that are of structural steel framing. The brick arches were typically covered with a very low-strength concrete fill, usually unreinforced. In many instances, various archaic diaphragm systems were patented by contractors.

5.8.5.2 Stiffness for Analysis

A. Linear Static Procedure

Existing archaic diaphragm systems are modeled as a horizontal diaphragm with equivalent thickness of arches and concrete fill. Development of truss elements between steel beams and compression elements of arches could also be considered. Flexibility of archaic diaphragms should be considered in the distribution of lateral loads to vertical elements, especially if spans are large.

All preceding comments for existing diaphragms apply for archaic diaphragms. Interaction of new and existing elements of strengthened elements (stiffness compatibility) must be considered in cases where steel trusses are added as part of a seismic upgrade. Load transfer mechanisms between new and existing diaphragm elements must be considered in determining the flexibility of the strengthened diaphragm.

For modeling assumptions and limitations, see the preceding comments related to force transfer, and interaction between diaphragm elements. Analyses are required to verify that elastic diaphragm response assumptions are valid.

B. Nonlinear Static Procedure

Inelastic properties of archaic diaphragms should be chosen with caution for seismic analyses. For the case of archaic diaphragms, inelastic models similar to those of archaic timber diaphragms in unreinforced masonry buildings may be appropriate. Inelastic deformation limits of archaic diaphragms should be lower than those prescribed for a concrete-filled diaphragm.

5.8.5.3 Strength and Deformation Acceptance Criteria

Member capacities of archaic diaphragm components can be calculated assuming little or no tension capacity except for the steel beam members. Gravity force effects must be included in the calculations for all components of these diaphragms. Force transfer mechanisms between various members and between frame elements must be considered to verify the completion of the load path.

In many cases, diaphragm distress could result in life safety considerations, due to possible loss of bearing support for the elements of the arches. Goals for higher performance would limit the amount of diagonal tension stresses, to insure that the load transfer mechanism was still complete. Deformations should be limited below the threshold of deflections that cause damage to other elements (either structural or nonstructural) at specified Performance Levels. These values must be established in conjunction with those for steel frames. Archaic diaphragm components should be considered as force-limited, so Equation 3-19 must be used.

5.8.5.4 Rehabilitation Measures

See the *Commentary.*

5.8.6 Chord and Collector Elements

5.8.6.1 General

Chords and collectors for all the previously described diaphragms typically consist of the steel framing that supports the diaphragm. When structural concrete is present, additional slab reinforcing may act as the chord or collector for tensile loads, while the slab carries chord or collector compression. When the steel framing acts as a chord or collector, it is typically attached to the deck with spot welds or by mechanical fasteners. When reinforcing acts as the chord or collector, load transfer occurs through bond between the reinforcing bars and the concrete.

5.8.6.2 Stiffness for Analysis

Modeling assumptions similar to those for equivalent frame members should be used.

5.8.6.3 Strength and Deformation Acceptance Criteria

Capacities of chords and collectors are provided by the AISC LRFD Specifications (1994a) and ACI-318 (ACI, 1995; see Chapter 6 for the citation) design guides. Inelastic action may occur, depending on the configuration of the diaphragm. It is desirable to design chord and collector components for a force that will develop yielding or ductile failure in either the diaphragm or vertical lateral-force-resisting system, so that the chords and collectors are not the weak link in the load path. In some cases, failure of chord and collector components may result in a life safety consideration when beams act as the chords or collectors and vertical support is compromised. Goals for higher performance would limit stresses and damage in chords and collectors, keeping the load path intact.

In buildings where the steel framing members that support the diaphragm act as collectors, the steel components may be alternately in tension and compression. If all connections to the diaphragm are sufficient, the diaphragm will prevent buckling of the chord member so values of m equal to 1, 6, and 8 may be used for IO, LS, and CP, respectively. If the diaphragm provides only limited support against buckling of the chord or collector, values of m equal to 1, 2, and 3 should be used. Where chords or collectors carry gravity loads along with seismic loads, they should be checked as members with combined loading using Equations 5-10 and 5-11. Welds and connectors joining the diaphragms to the collectors should be considered to be force-controlled.

5.8.6.4 Rehabilitation Measures

See the *Commentary.*

5.9 Steel Pile Foundations

5.9.1 General

Steel piles are one of the most common components for building foundations. Wide flange shapes (H piles) or structural tubes, with and without concrete infills, are the most commonly used shapes. Piles are usually driven in groups. A reinforced concrete pile cap is then cast over each group, and a steel column with a base plate is attached to the pile cap with anchor bolts.

The piles provide strength and stiffness to the foundation in one of two ways. Where very strong soil or rock lies at not too great a distance below the building site, the pile forces are transferred directly to the soil or rock at the bearing surface. Where this condition is not met, the piles are designed to transfer their load to the soil through friction. The design of the entire foundation is covered in Chapter 4 of these *Guidelines*. The design of the steel piles is covered in the following subsections.

5.9.2 Stiffness for Analysis

If the pile cap is below grade, the foundation attains much of its stiffness from the pile cap bearing against the soil. Equivalent soil springs may be derived as discussed in Chapter 4. The piles may also provide significant stiffness through bending and bearing against the soil. The effective pile contribution to stiffness is decreased if the piles are closely spaced; this group effect must be taken into account when calculating foundation and strength. For a more detailed description, see the *Commentary,* Section C5.9.2, and Chapter 4 of these *Guidelines.*

5.9.3 Strength and Deformation Acceptance Criteria

Buckling of steel piles is not a concern, since the soil provides lateral support. The moments in the piles may be calculated in one of two ways. The first is an elastic method that requires finding the effective point of fixity; the pile is then designed as a cantilever column. The second, a nonlinear method, requires a computer program that is available at no cost. Details are given in the *Commentary,* Section C5.9.2.

Once the axial force and maximum bending moments are known, the pile strength acceptance criteria are the same as for a steel column, as given in Equation 5-10. The expected axial and flexural strengths in Equation 5-10 are computed for an unbraced length equal to zero. Note that Equation 5-11 does not apply to steel piles. Exceptions to these criteria, where liquefaction is a concern, are discussed in the *Commentary,* Section C5.9.2.

5.9.4 Rehabilitation Measures for Steel Pile Foundations

Rehabilitation of the pile cap is covered in Chapter 6. Chapter 4 covers general criteria for the rehabilitation of the foundation element. In most cases, it is not possible to rehabilitate the existing piles. Increased stiffness and strength may be gained by driving additional piles near existing groups and then adding a new pile cap. Monolithic behavior can be gained by connecting the new and old pile caps with epoxied dowels, or other means.

5.10 Definitions

Beam: A structural member whose primary function is to carry loads transverse to its longitudinal axis; usually a horizontal member in a seismic frame system.

Braced frame: An essentially vertical truss system of concentric or eccentric type that resists lateral forces.

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

Connection: A link between components or elements that transmits actions from one component or element to another component or element. Categorized by type of action (moment, shear, or axial), connection links are frequently nonductile.

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

Diagonal bracing: Inclined structural members carrying primarily axial load, employed to enable a structural frame to act as a truss to resist horizontal loads.

Dual system: A structural system included in buildings with the following features:

- An essentially complete space frame provides support for gravity loads.
- Resistance to lateral load is provided by concrete or steel shear walls, steel eccentrically braced frames (EBF), or concentrically braced frames (CBF) along with moment-resisting frames (Special Moment Frames, or Ordinary Moment Frames) that are capable of resisting at least 25% of the lateral loads.
- Each system is also designed to resist the total lateral load in proportion to its relative rigidity.

Eccentric braced frame (EBF): A diagonal braced frame in which at least one end of each diagonal bracing member connects to a beam a short distance from either a beam-to-column connection or another brace end.

Joint: An area where two or more ends, surfaces, or edges are attached. Categorized by the type of fastener or weld used and the method of force transfer.

Lateral support member: A member designed to inhibit lateral buckling or lateral-torsional buckling of a component.

Link: In an EBF, the segment of a beam that extends from column to brace, located between the end of a diagonal brace and a column, or between the ends of two diagonal braces of the EBF. The length of the link is defined as the clear distance between the diagonal brace and the column face, or between the ends of two diagonal braces.

Link intermediate web stiffeners: Vertical web stiffeners placed within the link.

Link rotation angle: The angle of plastic rotation between the link and the beam outside of the link derived using the specified base shear, V.

LRFD (Load and Resistance Factor Design): A method of proportioning structural components (members, connectors, connecting elements, and assemblages) using load and resistance factors such that no applicable limit state is exceeded when the structure is subjected to all design load combinations.

Moment frame: A building frame system in which seismic shear forces are resisted by shear and flexure in members and joints of the frame.

Nominal strength: The capacity of a structure or component to resist the effects of loads, as determined by (1) computations using specified material strengths and dimensions, and formulas derived from accepted principles of structural mechanics, or (2) field tests or laboratory tests of scaled models, allowing for modeling effects, and differences between laboratory and field conditions.

Ordinary Moment Frame (OMF): A moment frame system that meets the requirements for Ordinary Moment Frames as defined in seismic provisions for new construction in AISC (1994a), Chapter 5.

P-Δ effect: The secondary effect of column axial loads and lateral deflection on the shears and moments in various components of a structure.

Panel zone: The area of a column at the beam-to-column connection delineated by beam and column flanges.

Required strength: The load effect (force, moment, stress, as appropriate) acting on a component or connection, determined by structural analysis from the factored loads (using the most appropriate critical load combinations).

Resistance factor: A reduction factor applied to member resistance that accounts for unavoidable deviations of the actual strength from the nominal value, and the manner and consequences of failure.

Slip-critical joint: A bolted joint in which slip resistance of the connection is required.

Special Moment Frame (SMF): A moment frame system that meets the special requirements for frames as defined in seismic provisions for new construction.

Structural system: An assemblage of load-carrying components that are joined together to provide regular interaction or interdependence.

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

X-braced frame: A concentric braced frame (CBF) in which a pair of diagonal braces crosses near the mid-length of the braces.

Y-braced frame: An eccentric braced frame (EBF) in which the stem of the Y is the link of the EBF system.

5.11 Symbols

This list may not contain symbols defined at their first use if not used thereafter.

A_b	Gross area of bolt or rivet, in.^{2}
A_c	Rivet area, in.^{2}
A_e	Effective net area, in.^{2}
A_f	Flange area of member, in.^{2}
A_g	Gross area, in.^{2}
A_{st}	Area of link stiffener, in.^{2}
A_w	Effective area of weld, in.^{2}
C_b	Coefficient to account for effect of nonuniform moment; given in AISC (1994a)
E	Young's modulus of elasticity, 29,000 ksi
F_{EXX}	Classification strength of weld metal, ksi
F_{te}	Expected tensile strength, ksi
F_v	Design shear strength of bolts or rivets, ksi
F_y	Specified minimum yield stress for the type of steel being used, ksi
F_{yb}	F_y of a beam, ksi
F_{yc}	F_y of a column, ksi
F_{ye}	Expected yield strength, ksi
F_{yf}	F_y of a flange, ksi
G	Shear modulus of steel, 11,200 ksi
I_b	Moment of inertia of a beam, in.^{4}
I_c	Moment of inertia of a column
K	Length factor for brace (see AISC, 1994a)
K_e	Stiffness of a link beam, kip/in.
K_s	Rotational stiffness of a connection, kip-in./rad
K_w	Stiffness of wall, kip/in.
K_θ	Rotational stiffness of a partially restrained connection, kip-in./rad
L	Length of bracing member, in.
L_p	The limiting unbraced length between points of lateral restraint for the full plastic moment capacity to be effective (see AISC, 1994a)
L_r	The limiting unbraced length between points of lateral support beyond which elastic lateral torsional buckling of the beam is the failure mode (see AISC, 1994a)
M_{CE}	Expected flexural strength of a member or joint, kip-in.
M_{CEx}	Expected bending strength of a member about the x-axis, kip-in.
M_{CEy}	Expected bending strength of a member about y-axis, kip-in.
M_p	Plastic bending moment, kip-in.
M_x	Bending moment in a member for the x-axis, kip-in.
M_y	Bending moment in a member for the y-axis, kip-in.
N_b	Number of bolts or rivets
P	Axial force in a member, kips
PR	Partially restrained
P_{cr}	Critical compression strength of bracing, kips
P_{CL}	Lower-bound axial strength of column, kips
P_u	Required axial strength of a column or a link, kips
P_{ye}	Expected yield axial strength of a member = $F_{ye}A_g$, kips
Q_{CE}	Expected strength of a component or element at the deformation level under consideration in a deformation-controlled action
Q_{CL}	Lower-bound estimate of the strength of a component or element at the deformation level under consideration for a force-controlled action
V_{CE}	Expected shear strength of a member, kips
V_{CE}	Shear strength of a link beam, kips
V_{ya}	Nominal shear strength of a member modified by the axial load magnitude, kips
Z	Plastic section modulus, in.^{3}
a	Clear width of wall between columns

b Width of compression element, in.
b_a Connection dimension
b_{cf} Column flange width, in.
b_f Flange width, in.
b_t Connection dimension
d Overall depth of member, in.
d_b Overall beam depth, in.
d_c Overall column depth, in.
d_v Bolt or rivet diameter, in.
d_z Overall panel zone depth between continuity plates, in.
e EBF link length, in.
h Average story height above and below a beam-column joint
h Clear height of wall between beams
h Distance from inside of compression flange to inside of tension flange, in.
h_c Assumed web depth for stability, in.
h_v Height of story v
k_v Shear buckling coefficient
l_b Length of beam
l_c Length of column
m A modification factor used in the acceptance criteria of deformation-controlled components or elements, indicating the available ductility of a component action
m_e Effective m
m_x Value of m for bending about x-axis of a member
m_y Value of m for bending about y-axis of a member
r Governing radius of gyration, in.
r_y Radius of gyration about y axis, in.
t Thickness of link stiffener, in.
t_a Thickness of angle, in.
t_{bf} Thickness of beam flange, in.
t_{cf} Thickness of column flange, in.
t_f Thickness of flange, in.
t_p Thickness of panel zone including doubler plates, in.
t_p Thickness of flange plate, in.
t_w Thickness of web, in.
t_w Thickness of plate wall
t_z Thickness of panel zone (doubler plates not necessarily included), in.
w Length of flange angle
w_z Width of panel zone between column flanges, in.

Δ Generalized deformation, unitless
Δ_i Inter-story displacement (drift) of story i divided by the story height
Δ_y Generalized yield deformation, unitless

θ Generalized deformation, radians
θ_i Inter-story drift ratio, radians
θ_y Generalized yield deformation, radians
κ A reliability coefficient used to reduce component strength values for existing components based on the quality of knowledge about the components' properties (see Section 2.7.2)
λ Slenderness parameter
λ_p Limiting slenderness parameter for compact element
λ_r Limiting slenderness parameter for noncompact element
ρ Ratio of required axial force (P_u) to nominal shear strength (V_y) of a link
ρ_{lp} Yield deformation of a link beam
ϕ Resistance factor = 1.0

5.12 References

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6. Concrete (Systematic Rehabilitation)

6.1 Scope

Engineering procedures for estimating the seismic performance of lateral-force-resisting concrete components and elements are described in this chapter. Methods are applicable for concrete components that are either (1) existing components of a building system, (2) rehabilitated components of a building system, or (3) new components that are added to an existing building system.

Information needed for Systematic Rehabilitation of concrete buildings, as described in Chapter 2 and Chapter 3, is presented herein. Symbols used exclusively in this chapter are defined in Section 6.15. Section 6.2 provides a brief historical perspective of the use of concrete in building construction; a comprehensive historical perspective is contained in the *Commentary*. In Section 6.3, material and component properties are discussed in detail. Important properties of in-place materials and components are described in terms of physical attributes as well as how to determine and measure them. Guidance is provided on how to use the values in Tables 6-1 to 6-3 that might be used as default assumptions for material properties in a preliminary analysis.

General analysis and design assumptions and requirements are covered in Section 6.4. Critical modes of failure for beams, columns, walls, diaphragms, and foundations are discussed in terms of shear, bending, and axial forces. Components that are usually controlled by deformation are described in general terms. Other components that have limiting behavior controlled by force levels are presented along with Analysis Procedures.

Sections 6.5 through 6.13 cover the majority of the various structural concrete elements, including frames, braced frames, shear walls, diaphragms, and foundations. Modeling procedures, acceptance criteria, and rehabilitation measures for each component are discussed.

6.2 Historical Perspective

The components of concrete seismic resisting elements are columns, beams, slabs, braces, collectors, diaphragms, shear walls, and foundations. There has been a constant evolution in form, function, concrete strength, concrete quality, reinforcing steel strength, quality and detailing, forming techniques, and concrete placement techniques. All of these factors have a significant impact on the seismic resistance of a concrete building. Innovations such as prestressed and precast concrete, post tensioning, and lift slab construction have created a multivariant inventory of existing concrete structures.

The practice of seismic resistant design is relatively new to most areas of the United States, even though such practice has been evolving in California for the past 70 years. It is therefore important to investigate the local practices relative to seismic design when trying to analyze a specific building. Specific benchmark years can generally be determined for the implementation of seismic resistant design in most locations, but caution should be exercised in assuming optimistic characteristics for any specific building.

Particularly with concrete materials, the date of original building construction has significant influence on seismic performance. In the absence of deleterious conditions or materials, concrete gains compressive strength from the time it is originally cast and in-place. Strengths typically exceed specified design values (28-day or similar). Early uses of concrete did not specify any design strength, and low-strength concrete was not uncommon. Also, early use of concrete in buildings often employed reinforcing steel with relatively low strength and ductility, limited continuity, and reduced bond development. Continuity between specific existing components and elements (e.g., beams and columns, diaphragms and shear walls) is also particularly difficult to assess, given the presence of concrete cover and other barriers to inspection. Also, early use of concrete was expanded by use of proprietary structural system designs and construction techniques. Some of these systems are described in the *Commentary* Section C6.2. The design professional is cautioned to fully examine available construction documents and in-place conditions in order to properly analyze and characterize historical concrete elements and components of buildings.

As indicated in Chapter 1, great care should be exercised in selecting the appropriate rehabilitation approaches and techniques for application to historic

Table 6-1 Tensile and Yield Properties of Concrete Reinforcing Bars for Various Periods

		Structural[1]	Intermediate[1]	Hard[1]			
	Grade	33	40	50	60	70	75
Year[3]	Minimum Yield[2] (psi)	33,000	40,000	50,000	50,000	60,000	75,000
	Minimum Tensile[2] (psi)	55,000	70,000	80,000	90,000	80,000	100,000
1911-1959		x	x	x			
1959-1966		x	x	x	x		x
1966-1972			x	x	x		
1972-1974			x	x	x		
1974-1987			x	x	x	x	
1987-present			x	x	x	x	x

General Note: An entry “x” indicates the grade was available in those years.

Specific Notes: 1. The terms structural, intermediate, and hard became obsolete in 1968.
2. Actual yield and tensile strengths may exceed minimum values.
3. Until about 1920, a variety of proprietary reinforcing steels were used. Yield strengths are likely to be in the range from 33,000 psi to 55,000 psi, but higher values are possible Plain and twisted square bars were sometimes used between 1900 and 1949.

buildings in order to preserve their unique characteristics.

Tables 6-1 and 6-2 contain a summary of reinforcing steel properties that might be expected to be encountered. Table 6-1 provides a range of properties for use where only the year of construction is known. Where both ASTM designations and year of construction are known, use Table 6-2. Properties of Welded Wire Fabric for various periods of construction can be obtained from the Wire Reinforcement Institute. Possible concrete strengths as a function of time are given in Table 6-3. A more detailed historical treatment is provided in Section C6.2 of the *Commentary*, and the reader is encouraged to review the referenced documents..

6.3 Material Properties and Condition Assessment

6.3.1 General

Quantification of in-place material properties and verification of existing system configuration and condition are necessary to analyze a building properly. This section identifies properties requiring consideration and provides guidelines for determining the properties of buildings. Also described is the need for a thorough condition assessment and utilization of knowledge gained in analyzing component and system behavior. Personnel involved in material property quantification and condition assessment shall be experienced in the proper implementation of testing practices, and interpretation of results.

The extent of in-place materials testing and condition assessment needed is related to the availability and accuracy of construction (as-built) records, quality of materials and construction, and physical condition. Documentation of properties and grades of material used in component/connection construction is invaluable and may be effectively used to reduce the amount of in-place testing required. The design professional is encouraged to research and acquire all available records from original construction.

6.3.2 Properties of In-Place Materials and Components

6.3.2.1 Material Properties

Mechanical properties of component and connection material strongly influence the structural behavior under load. Mechanical properties of greatest interest for concrete elements and components include the following:

- Concrete compressive and tensile strengths and modulus of elasticity

Table 6-2 Tensile and Yield Properties of Concrete Reinforcing Bars for Various ASTM Specifications and Periods

				Structural[1]	**Intermediate[1]**	**Hard[1]**			
			Grade	**33**	**40**	**50**	**60**	**70**	**75**
			Minimum Yield[2] (psi)	33,000	40,000	50,000	50,000	60,000	75,000
ASTM	Steel Type	Year Range[3]	Minimum Tensile[2] (psi)	55,000	70,000	80,000	90,000	80,000	100,000
A15	Billet	1911-1966		x	x	x			
A16	Rail[4]	1913-1966				x			
A61	Rail[4]	1963-1966					x		
A160	Axle	1936-1964		x	x	x			
A160	Axle	1965-1966		x	x	x	x		
A408	Billet	1957-1966		x	x	x			
A431	Billet	1959-1966							x
A432	Billet	1959-1966					x		
A615	Billet	1968-1972			x		x		x
A615	Billet	1974-1986			x		x		
A615	Billet	1987-1997			x		x		x
A616	Rail[4]	1968-1997				x	x		
A617	Axle	1968-1997			x		x		
A706	Low-Alloy[5]	1974-1997						x	
A955	Stainless	1996-1997			x		x		x

General Note: An entry “x” indicates the grade was available in those years.

Specific Notes:
1. The terms structural, intermediate, and hard became obsolete in 1968.
2. Actual yield and tensile strengths may exceed minimum values.
3. Until about 1920, a variety of proprietary reinforcing steels were used. Yield strengths are likely to be in the range from 33,000 psi to 55,000 psi, but higher values are possible Plain and twisted square bars were sometimes used between 1900 and 1949.
4. Rail bars should be marked with the letter “R.” Bars marked “s!” (ASTM 616) have supplementary requirements for bend tests.
5. ASTM steel is marked with the letter “W.”

Table 6-3 Compressive Strength of Structural Concrete (psi)[1]

Time Frame	Footings	Beams	Slabs	Columns	Walls
1900–1919	1000–2500	2000–3000	1500–3000	1500–3000	1000–2500
1920–1949	1500–3000	2000–3000	2000–3000	2000–4000	2000–3000
1950–1969	2500–3000	3000–4000	3000–4000	3000–6000	2500–4000
1970–Present	3000–4000	3000–5000	3000–5000	3000–10000[2]	3000–5000

1. Concrete strengths are likely to be highly variable within any given older structure.
2. Exceptional cases of very high strength concrete may be found.

- Yield and ultimate strength of conventional and prestressing reinforcing steel and metal connection hardware
- Ductility, toughness, and fatigue properties
- Metallurgical condition of the reinforcing steel, including carbon equivalent, presence of any degradation such as corrosion, bond with concrete, and chemical composition.

The effort required to determine these properties depends on the availability of accurate updated construction documents and drawings, quality and type of construction (absence of degradation), accessibility, and condition of materials. The method of analysis (e.g., Linear Static Procedure, Nonlinear Static Procedure) to be used in the rehabilitation may also influence the scope of the testing

In general, the determination of material properties (other than connection behavior) is best accomplished through removal of samples and laboratory analysis. Sampling shall take place in primary gravity- and lateral-force-resisting components. Where possible, sampling shall occur in regions of reduced stress to limit the effects of reduced sectional area. The size of the samples and removal practices to be followed are referenced in the *Commentary*. The frequency of sampling, including the minimum number of tests for property determination, is addressed in Section 6.3.2.4.

Generally, mechanical properties for both concrete and reinforcing steel can be established from combined core and specimen sampling at similar locations, followed by laboratory testing. For concrete, the sampling program shall consist of the removal of standard vertical or horizontal cores. Core drilling shall be preceded by nondestructive location of the reinforcing steel, and shall avoid damaging the existing reinforcing steel as much as practicable. Core holes shall be filled with comparable-strength concrete or grout. For conventional reinforcing and bonded prestressing steel, sampling shall consist of the removal of local bar segments (extreme care shall be taken with removal of any prestressing steels). Depending on the location and amount of bar removed, replacement spliced material shall be installed to maintain continuity.

6.3.2.2 Component Properties

Structural elements often utilize both primary and secondary components to perform their load- and deformation-resisting function. Behavior of the components, including beams, columns, and walls, is dictated by such properties as cross-sectional dimensions and area, reinforcing steel location, width-to-thickness and slenderness ratios, lateral buckling resistance, and connection details. This behavior may also be altered by the presence of degradation or physical damage. The following component properties shall be established during the condition assessment phase of the seismic rehabilitation process to aid in evaluating component behavior (see Section 6.3.3 for assessment guidelines):

- Original and current cross-sectional dimensions
- As-built configuration and physical condition of primary component end connections, and intermediate connections such as those between diaphragms and supporting beams/girders
- Size, anchorage, and thickness of other connector materials, including metallic anchor bolts, embeds, bracing components, and stiffening materials, commonly used in precast and tilt-up construction

- Characteristics that may influence the continuity, moment-rotation, or energy dissipation and load transfer behavior of connections
- Confirmation of load transfer capability at component-to-element connections, and overall element/structure behavior

These properties may be needed to characterize building performance properly in the seismic analysis. The starting point for assessing component properties and condition should be retrieval of available construction documents. Preliminary review of these documents shall be performed to identify primary vertical- (gravity) and lateral load-carrying elements and systems, and their critical components and connections. In the absence of a complete set of building drawings, the design professional must perform a thorough inspection of the building to identify these elements, systems, and components as indicated in Section 6.3.3.

In the absence of degradation, component dimensions and properties from original drawings may be used in structural analyses without introducing significant error. Variance from nominal dimensions, such as reinforcing steel size and effective area, is usually small.

6.3.2.3 Test Methods to Quantify Properties

To obtain the desired in-place mechanical properties of materials and components, it is necessary to use proven destructive and nondestructive testing methods. Certain field tests—such as estimation of concrete compressive strength from hardness and impact resistance tests—may be performed, but laboratory testing shall be used where strength is critical. Critical properties of concrete commonly include the compressive and tensile strength, modulus of elasticity, and unit weight. Samples of concrete and reinforcing and connector steel shall also be examined for physical condition (see Section 6.3.3.2).

Accurate determination of existing concrete strength properties is typically achieved through removal of core samples and performance of laboratory destructive testing. Removal of core samples should employ the procedures contained in *ASTM C 42*. Testing should follow the procedures contained in *ASTM C 42*, *C 39*, and *C 496*. The measured strength from testing must be correlated to in-place concrete compressive strength; the *Commentary* provides further guidance on correlating core strength to in-place strength and other recommendations. The *Commentary* provides references for various test methods that may be used to estimate material properties.

Accurate determination of existing reinforcing steel strength properties is typically achieved through removal of bar or tendon length samples and performance of laboratory destructive testing. The primary strength measures for reinforcing and prestressing steels are the tensile yield strength and ultimate strength, as used in the structural analysis. Strength values may be obtained by using the procedures contained in *ASTM A 370*. Prestressing materials must also meet the supplemental requirements in *ASTM A 416*, *A 421*, or *A 722*, depending on material type. The chemical composition may also be determined from the retrieved samples.

Particular test methods that may be used for connector steels include wet and dry chemical composition tests, and direct tensile and compressive strength tests. For each test, industry standards published by ASTM, including Standard *A 370*, exist and shall be followed. For embedded connectors, the strength of the material may also be assessed in situ using the provisions of *ASTM E 488*. The *Commentary* provides references for these tests.

Usually, the reinforcing steel system used in construction of a specific building is of a common grade and strength. Occasionally one grade of reinforcement is used for small-diameter bars (e.g., those used for stirrups and hoops) and another grade for large-diameter bars (e.g., those used for longitudinal reinforcement). Furthermore, it is possible that a number of different concrete design strengths (or "classes") have been employed. In developing a testing program, the design professional shall consider the possibility of varying concrete classes. Historical research and industry documents also contain insight on material mechanical properties used in different construction eras. Section 6.3.2.5 provides strength data for most primary concrete and reinforcing steels used. This information, with laboratory and field test data, may be used to gain confidence in in situ strength properties.

6.3.2.4 Minimum Number of Tests

In order to quantify in-place properties accurately, it is important that a minimum number of tests be conducted on primary components in the lateral-force-resisting system. As stated previously, the minimum number of tests is dictated by available data from original construction, the type of structural system employed, desired accuracy, and quality/condition of in-place materials. The accessibility of the structural system may also influence the testing program scope. The focus of this testing shall be on primary lateral-force-resisting components and on specific properties needed for analysis. The test quantities provided in this section are minimum numbers; the design professional should determine whether further testing is needed to evaluate as-built conditions.

Testing is not required on components other than those of the lateral-force-resisting system. If the existing lateral-force-resisting system is being replaced in the rehabilitation process, minimum material testing is needed to qualify properties of existing materials at new connection points.

A. Concrete Materials

For each concrete element type (such as a shear wall), a minimum of three core samples shall be taken and subjected to compression tests. A minimum of six tests shall be done for the complete concrete building structure, subject to the limitations noted below. If varying concrete classes/grades were employed in building construction, a minimum of three samples and tests shall be performed for each class. Test results shall be compared with strength values specified in the construction documents. The core strength shall be converted to in situ concrete compressive strength (f'_c) as in Section C6.3.2.3 of the *Commentary.* The unit weight and modulus of elasticity shall be derived or estimated during strength testing. Samples should be taken at random locations in components critical to structural behavior of the building. Tests shall also be performed on samples from components that are damaged or degraded, to quantify their condition. If test values less than the specified strength in the construction documents are found, further strength testing shall be performed to determine the cause or identify whether the condition is localized.

The minimum number of tests to determine compressive and tensile strength shall also conform to the following criteria.

- For concrete elements for which the specified design strength is known and test results are not available, a minimum of three cores/tests shall be conducted for each floor level, 400 cubic yards of concrete, or 10,000 square feet of surface area (estimated smallest of the three).
- For concrete elements for which the design strength is unknown and test results are not available, a minimum of six cores/tests shall be conducted for each floor level, 400 cubic yards of concrete, or 10,000 square feet of surface area (use smallest number). Where the results indicate that different classes of concrete were employed, the degree of testing shall be increased to confirm class use.
- A minimum of three samples shall be removed for splitting tensile strength determination, if a lightweight aggregate concrete were used for primary components. Additional tests may be warranted, should the coefficient of variation in test results exceed 14%.

If a sample population greater than the minimum specified is used in the testing program and the coefficient of variation in test results is less than 14%, the mean strength derived may be used as the expected strength in the analysis. If the coefficient of variation from testing is greater than 14%, additional sampling and testing should be performed to improve the accuracy of testing or understanding of in situ material strength. The design professional (and subcontracted testing agency) shall carefully examine test results to verify that suitable sampling and testing procedures were followed, and that appropriate values for the analysis were selected from the data. In general, the expected concrete strength shall not exceed the mean less one standard deviation in situations where variability is greater than 14%.

In addition to destructive sampling and testing, further quantification of concrete strength may be estimated via ultrasonics, or another nondestructive test method (see the *Commentary*). Because these methods do not yield accurate strength values directly, they should be used for confirmation and comparison only and shall not be substituted for core sampling and laboratory testing.

B. Conventional Reinforcing and Connector Steels

In terms of defining reinforcing and connector steel strength properties, the following guidelines shall be followed. Connector steel is defined as additional structural or bolting steel material used to secure precast and other concrete shapes to the building structure. Both yield and ultimate strengths shall be determined. A minimum of three tensile tests shall be conducted on conventional reinforcing steel samples from a building for strength determination, subject to the following supplemental conditions.

- If original construction documents defining properties exist, and if an Enhanced Rehabilitation Objective (greater than the BSO) is desired, at least three strength coupons shall be randomly removed from each element or component type (e.g., slabs, walls, beams) and tested.

- If original construction documents defining properties do not exist, but the approximate date of construction is known and a common material grade is confirmed (e.g., all bars are Grade 60 steel), at least three strength coupons shall be randomly removed from each element or component type (e.g., beam, wall) for every three floors of the building. If the date of construction is unknown, at least six such samples/tests, for every three floors, shall be performed. This is required to satisfy the BSO.

All sampled steel shall be replaced with new fully spliced and connected material, unless an analysis confirms that replacement of function is not required.

C. Prestressing Steels

The sampling of prestressing steel tendons for laboratory testing shall be accomplished with extreme care; only those prestressed components that are a part of the lateral-force-resisting system shall be considered. Components in diaphragms should generally be excluded from testing. If limited information exists regarding original materials and the prestressing force applied, the design professional must attempt to quantify properties for analysis. Tendon removals shall be avoided if possible in prestressed members. Only a minimum number of tendon samples for laboratory testing shall be taken.

Determination of material properties may be possible, without tendon removal or prestress removal, by careful sampling of either the tendon grip or extension beyond the anchorage.

All sampled steel shall be replaced with new fully connected and stressed material and anchorage hardware unless an analysis confirms that replacement of function is not required.

D. General

For other material properties, such as hardness and ductility, no minimum number of tests is prescribed. Similarly, standard test procedures may not exist. The design professional shall examine the particular need for this type of testing and establish an adequate protocol. In general, it is recommended that a minimum of three tests be conducted to determine any property. If outliers (results with coefficients of variation greater than 15%) are detected, additional tests shall be performed until an accurate representation of the property is gained.

6.3.2.5 Default Properties

Mechanical properties for materials and components shall be based on available historical data for the particular structure and tests on in-place conditions. Should extenuating circumstances prevent minimum material sampling and testing from being performed, default strength properties may be used. Default material and component properties have been established for concrete compressive strength and reinforcing steel tensile and yield strengths from published literature; these are presented in Tables 6-1 to 6-3. These default values are generally conservative, representing values reduced from mean strength in order to address variability. However, the selection of a default strength for concrete shall be made with care because of the multitude of mix designs and materials used in the construction industry.

For concrete default compressive strength, lower-bound values from Table 6-3 may be used. The default compressive strength shall be used to establish other strength and performance characteristics for the concrete as needed in the structural analysis.

Reinforcing steel tensile properties are presented in Tables 6-1 and 6-2. Because of lower variability, the lower-bound tabulated values may be used without further reduction. For Rehabilitation Objectives in which default values are assumed for existing reinforcing steel, no welding or mechanical coupling of new reinforcing to the existing reinforcing steel is permitted. For buildings constructed prior to 1950, the bond strength developed between reinforcing steel and concrete may be less than present-day strength. The tensile lap splices and development length of older plain reinforcing should be considered as 50% of the capacity of present-day tabulated values (such as in *ACI 318-95*

[ACI, 1995]) unless further justified through testing and assessment (CRSI, 1981).

For connector materials, the nominal strength from design and construction documents may be used. In the absence of this information, the default yield strength for steel connector material may be taken as 27,000 psi.

Default values for prestressing steel in prestressed concrete construction shall not be used, unless circumstances prevent material sampling/testing from being performed. In this case, it may be prudent to add a new lateral-force-resisting system to the building.

6.3.3 Condition Assessment

6.3.3.1 General

A condition assessment of the existing building and site conditions shall be performed as part of the seismic rehabilitation process. The goal of this assessment is threefold:

- To examine the physical condition of primary and secondary components and the presence of any degradation
- To verify the presence and configuration of components and their connections, and the continuity of load paths between components, elements, and systems
- To review other conditions that may influence existing building performance, such as neighboring party walls and buildings, nonstructural components that may contribute to resistance, and any limitations for rehabilitation

The physical condition of existing components and elements, and their connections, must be examined for presence of degradation. Degradation may include environmental effects (e.g., corrosion, fire damage, chemical attack), or past or current loading effects (e.g., overload, damage from past earthquakes, fatigue, fracture). The condition assessment shall also examine for configurational problems observed in recent earthquakes, including effects of discontinuous components, construction deficiencies, poor fit-up, and ductility problems.

Component orientation, plumbness, and physical dimensions should be confirmed during an assessment. Connections in concrete components, elements, and systems require special consideration and evaluation. The load path for the system must be determined, and each connection in the load path(s) must be evaluated. This includes diaphragm-to-component and component-to-component connections. Where the connection is attached to one or more components that are expected to experience significant inelastic response, the strength and deformation capacity of connections must be evaluated. The condition and detailing of at least one of each connection type should be investigated.

The condition assessment also affords an opportunity to review other conditions that may influence concrete elements and systems, and overall building performance. Of particular importance is the identification of other elements and components that may contribute to or impair the performance of the concrete system in question, including infills, neighboring buildings, and equipment attachments. Limitations posed by existing coverings, wall and ceiling space, infills, and other conditions shall also be defined.

6.3.3.2 Scope and Procedures

The scope of the condition assessment should include all primary structural elements and components involved in gravity and lateral load resistance, as limited by accessibility. The knowledge and insight gained from the condition assessment is invaluable to the understanding of load paths and the ability of components to resist and transfer these loads. The degree of assessment performed also affects the κ factor that is used in the analysis, and the type of analysis (see Section 6.3.4).

A. Visual Inspection

Direct visual inspection provides the most valuable information, as it can be used to quickly identify any configurational issues, and it allows the measurement of component dimensions, and the determination whether degradation is present. The continuity of load paths may be established through viewing of components and connection condition. From visual inspection, the need for other test methods to quantify the presence and degree of degradation may be established. The dimensions of accessible primary components shall be measured and compared with available design information. Similarly, the configuration and condition of all connections (exposed surfaces) shall be verified with permanent deformations or other noted anomalies.

Industry-accepted procedures are cited in the *Commentary.*

Visual inspection of the specific building should include all elements and components constructed of concrete, including foundations, vertical and horizontal frame members, diaphragms (slabs), and connections. As a minimum, a representative sampling of at least 20% of the elements, components, and connections shall be visually inspected at each floor level. If significant damage or degradation is found, the assessment sample shall be increased to all critical components of similar type in the building. The damage should be quantified using supplemental methods cited in this chapter and the *Commentary.*

If coverings or other obstructions exist, indirect visual inspection through the obstruction may be conducted by using drilled holes and a fiberscope. If this method is not appropriate, then exposure will be necessary. Exposure is defined as local minimized removal of cover concrete and other materials to allow inspection of reinforcing system details; all damaged concrete cover shall be replaced after inspection. The following guidelines shall be used for assessing primary connections in the building.

- If detailed design drawings exist, exposure of at least three different primary connections shall occur, with the connection sample including different types (e.g., beam-column, column-foundation, beam-diaphragm). If no deviations from the drawings exist, the sample may be considered representative of installed conditions. If deviations are noted, then exposure of at least 25% of the specific connection type is necessary to identify the extent of deviation.

- In the absence of accurate drawings, exposure of at least three connections of each primary connection type shall occur for inspection. If common detailing is observed, this sample may be considered representative. If many different details of deviations are observed, increased connection inspection is warranted until an accurate understanding of building construction and behavior is gained.

B. Additional Testing

The physical condition of components and connectors may also dictate the need for certain destructive and nondestructive test methods. Such methods may be used to determine the degree of damage or presence of contamination, and to improve understanding of the internal condition and quality of the concrete. Further guidelines and procedures for destructive and nondestructive tests that may be used in the condition assessment are provided in the *Commentary.* The following paragraphs identify those nondestructive examination (NDE) methods having the greatest use and applicability to condition assessment.

- Surface NDE methods include infrared thermography, delamination sounding, surface hardness measurement, and crack mapping. These methods may be used to find surface degradation in components such as service-induced cracks, corrosion, and construction defects.

- Volumetric NDE methods, including radiography and ultrasonics, may be used to identify the presence of internal discontinuities, as well as to identify loss of section. Impact-echo ultrasonics is particularly useful because of ease of implementation and proven capability in concrete.

- Structural condition and performance may be assessed through on-line monitoring using acoustic emissions and strain gauges, and in-place static or dynamic load tests. Monitoring is used to determine if active degradation or deformations are occurring, while nondestructive load testing provides direct insight on load-carrying capacity.

- Locating, sizing, and initial assessment of the reinforcing steel may be completed using electromagnetic methods (such as pachometer). Further assessment of suspected corrosion activity should utilize electrical half-cell potential and resistivity measurements.

- Where it is absolutely essential, the level of prestress remaining in an unbonded prestressed system may be measured using lift-off testing (assuming original design and installation data are available), or another nondestructive method such as “coring stress relief” (ASCE, 1990).

The *Commentary* provides general background and references for these methods.

6.3.3.3 Quantifying Results

The results of the condition assessment shall be used in the preparation of building system models in the evaluation of seismic performance. To aid in this effort, the results shall be quantified, with the following specific topics addressed:

- Component section properties and dimensions
- Component configuration and presence of any eccentricities or permanent deformation
- Connection configuration and presence of any eccentricities
- Presence and effect of alterations to the structural system since original construction (e.g., doorways cut into shear walls)
- Interaction of nonstructural components and their involvement in lateral load resistance

As previously noted, the acceptance criteria for existing components depend on the design professional's knowledge of the condition of the structural system and material properties. All deviations noted between available construction records and as-built conditions shall be accounted for and considered in the structural analysis. Again, some removal of cover concrete is required during this stage to confirm reinforcing steel configuration.

Gross component section properties in the absence of degradation have been found to be statistically close to nominal. Unless concrete cracking, reinforcing corrosion, or other mechanisms are observed in the condition assessment to be causing damage or reduced capacity, the cross-sectional area and other sectional properties shall be taken as those from the design drawings. If some sectional material loss has occurred, the loss shall be quantified via direct measurement. The sectional properties shall then be reduced accordingly, using the principles of structural mechanics. If the degradation is significant, further analysis or rehabilitative measures shall be undertaken.

6.3.4 Knowledge (κ) Factor

As described in Section 2.7, computation of component capacities and allowable deformations involves the use of a knowledge (κ) factor. For cases where a Linear Static Procedure (LSP) will be used in the analysis, two possible values for κ exist (0.75 and 1.0). For nonlinear procedures, the design professional must obtain an in-depth understanding of the building structural system and condition to support the use of a κ factor of 1.0. This section further describes the requirements and selection criteria for a κ factor specific to concrete structural components.

If the concrete structural system is exposed and good access exists, significant knowledge regarding configuration and behavior may be gained through condition assessment. In general, a κ factor of 1.0 can be used when a thorough assessment is performed on the primary/secondary components and load paths, and the requirements of Sections 2.7 and 6.3.3 are met. This assessment should include exposure of at least one sample of each primary component connection type and comparison with construction documents. However, if original reinforcing steel shop drawings, material specifications, and field inspection or quality control records are available, this effort is not required.

If incomplete knowledge of as-built component or connection configuration exists because a smaller sampling is performed than that required for $\kappa = 1.0$, κ shall be reduced to 0.75. Rehabilitation requires that a minimum sampling be performed from which knowledge of as-built conditions can be surmised. Where a κ of 0.75 cannot be justified, no seismic resistance capacity may be used for existing components.

If all required testing for $\kappa = 1.0$ is done and the following situations prevail, κ shall be reduced to 0.75.

- Construction documents for the concrete structure are not available or are incomplete.
- Components are found degraded during assessment, for which further testing is required to qualify behavior and to use $\kappa = 1.0$.
- Components have high variability in mechanical properties (up to a coefficient of variation of 25%).
- Components shown in construction documents lack sufficient structural detail to allow proper analysis.
- Components contain archaic or proprietary material and their materials condition is uncertain.

6.3.5 Rehabilitation Issues

Upon determining that concrete elements in an existing building are deficient for the desired Rehabilitation Objective, the next step is to define rehabilitation or replacement alternatives. If replacement of the element is selected, design of the new element shall be in accordance with local building codes and the *NEHRP Recommended Provisions* for new buildings (BSSC, 1995).

6.3.6 Connections

Connections between existing concrete components and any components added to rehabilitate the original structure are critical to overall seismic performance. The design professional is strongly encouraged to examine as-built connections and perform any physical testing/inspection to assess their performance. All new connections shall be subject to the quality control provisions contained in these *Guidelines*. In addition, for connectors that are not cast-in-place, such as anchor bolts, a minimum of five samples from each connector type shall be tested after installation. Connectors that rely on ductility shall be tested according to Section 2.13. (See also Section 6.4.6.)

6.4 General Assumptions and Requirements

6.4.1 Modeling and Design

6.4.1.1 General Approach

Design approaches for an existing or rehabilitated building generally shall follow procedures of *ACI 318-95* (ACI, 1995), except as otherwise indicated in these *Guidelines*, and shall emphasize the following.

- Brittle or low-ductility failure modes shall be identified as part of the analysis. These typically include behavior in direct or nearly-direct compression, shear in slender components and in component connections, torsion in slender components, and reinforcement development, splicing, and anchorage. It is preferred that the stresses, forces, and moments acting to cause these failure modes be determined from consideration of the probable resistances at the locations for nonlinear action.

- Analysis of reinforced concrete components shall include an evaluation of demands and capacities at all sections along the length of the component. Particular attention shall be paid to locations where lateral and gravity loads produce maximum effects; where changes in cross section or reinforcement result in reduced strength; and where abrupt changes in cross section or reinforcement, including splices, may produce stress concentrations resulting in premature failure.

6.4.1.2 Stiffness

Component stiffnesses shall be calculated according to accepted principles of mechanics. Sources of flexibility shall include flexure, shear, axial load, and reinforcement slip from adjacent connections and components. Stiffnesses should be selected to represent the stress and deformation levels to which the components will be subjected, considering volume change effects (temperature and shrinkage) combined with design earthquake and gravity load effects.

A. Linear Procedures

Where design actions are determined using the linear procedures of Chapter 3, component effective stiffnesses shall correspond to the secant value to the yield point for the component, except that higher stiffnesses may be used where it is demonstrated by analysis to be appropriate for the design loading. The effective stiffness values in Table 6-4 should be used, except where little nonlinear behavior is expected or detailed evaluation justifies different values. These same stiffnesses may be appropriate for the initial stiffness for use in the nonlinear procedures of Chapter 3.

B. Nonlinear Procedures

Where design actions are determined using the nonlinear procedures of Chapter 3, component load-deformation response shall be represented by nonlinear load-deformation relations, except that linear relations are acceptable where nonlinear response will not occur in the component. The nonlinear load-deformation relation shall be based on experimental evidence or may be taken from quantities specified in Sections 6.5 through 6.13. The nonlinear load-deformation relation for the Nonlinear Static Procedure (NSP) may be composed of line segments or curves defining behavior under monotonically increasing lateral deformation. The nonlinear load-deformation relation for the Nonlinear Dynamic Procedure (NDP) may be composed of line segments or curves, and shall define behavior under monotonically increasing lateral deformation and under multiple reversed deformation cycles.

Figure 6-1 illustrates a generalized load-deformation relation that may be applicable for most concrete components evaluated using the NSP. The relation is

Table 6-4 Effective Stiffness Values

Component	Flexural Rigidity	Shear Rigidity	Axial Rigidity
Beams—nonprestressed	$0.5E_cI_g$	$0.4E_cA_w$	—
Beams—prestressed	E_cI_g	$0.4E_cA_w$	—
Columns in compression	$0.7E_cI_g$	$0.4E_cA_w$	E_cA_g
Columns in tension	$0.5E_cI_g$	$0.4E_cA_w$	E_sA_s
Walls—uncracked (on inspection)	$0.8E_cI_g$	$0.4E_cA_w$	E_cA_g
Walls—cracked	$0.5E_cI_g$	$0.4E_cA_w$	E_cA_g
Flat Slabs—nonprestressed	See Section 6.5.4.2	$0.4E_cA_g$	—
Flat Slabs—prestressed	See Section 6.5.4.2	$0.4E_cA_g$	—

Note: I_g for T-beams may be taken as twice the value of I_g of the web alone, or may be based on the effective width as defined in Section 6.4.1.3.
For shear stiffness, the quantity $0.4E_c$ has been used to represent the shear modulus G.

described by linear response from A (unloaded component) to an effective yield B. Subsequently, there is linear response, at reduced stiffness, from B to C, with sudden reduction in lateral load resistance to D, response at reduced resistance to E, and final loss of resistance thereafter. The slope from A to B shall be according to Section 6.4.1.2A. The slope from B to C, ignoring effects of gravity loads acting through lateral displacements, typically may be taken as equal to between zero and 10% of the initial slope. C has an ordinate equal to the strength of the component and an abscissa equal to the deformation at which significant strength degradation begins. It is permissible to represent the load-deformation relation by lines connecting points A, B, and C, provided that the calculated response is not beyond C. It is also acceptable to use more refined relations where they are justified by experimental evidence. Sections 6.5 through 6.13 recommend numerical values for the points identified in Figure 6-1.

Typically, the responses shown in Figure 6-1 are associated with flexural response or tension response. In this case, the resistance at $Q/Q_{CE} = 1.0$ is the yield value, and subsequent strain hardening accommodates strain hardening in the load-deformation relation as the member is deformed toward the expected strength. When the response shown in Figure 6-1 is associated with compression, the resistance at $Q/Q_{CE} = 1.0$ typically is the value at which concrete begins to spall, and strain hardening in well-confined sections may be associated with strain hardening of the longitudinal reinforcement and the confined concrete. When the

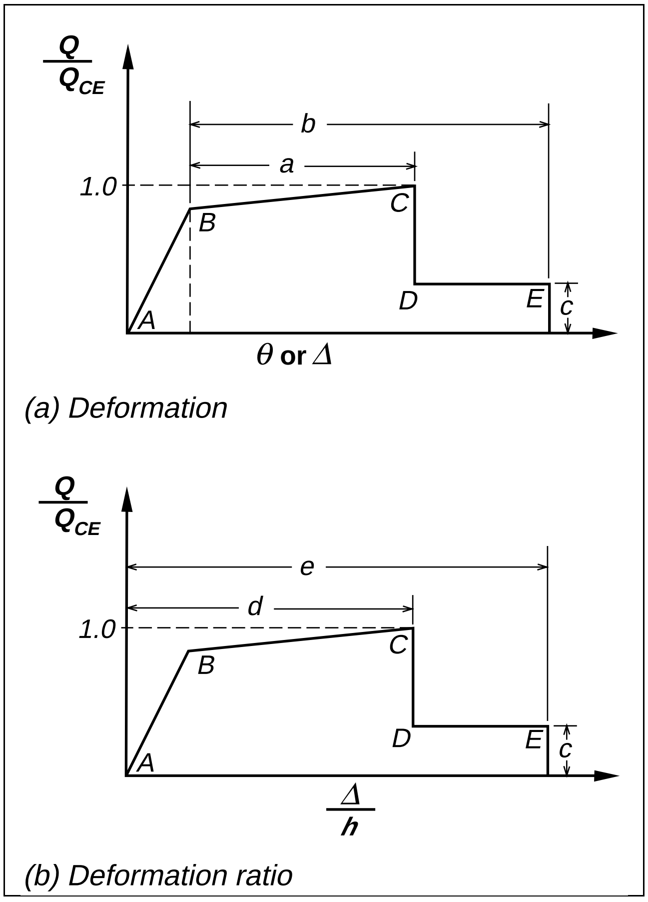

Figure 6-1 Generalized Load-Deformation Relation

response shown in Figure 6-1 is associated with shear, the resistance at Q/Q_{CE} = 1.0 typically is the value at which the design shear strength is reached, and no strain hardening follows.

Figure 6-1 shows two different ways to define the deformations, as follows:

(a) Deformation, or Type I. In this curve, deformations are expressed directly using terms such as strain, curvature, rotation, or elongation. The parameters *a* and *b* refer to those portions of the deformation that occur after yield; that is, the plastic deformation. The parameter *c* is the reduced resistance after the sudden reduction from *C* to *D*. Parameters *a*, *b*, and *c* are defined numerically in various tables in this chapter.

(b) Deformation Ratio, or Type II. In this curve, deformations are expressed in terms such as shear angle and tangential drift ratio. The parameters *d* and *e* refer to total deformations measured from the origin. Parameters *c*, *d*, and *e* are defined numerically in various tables in this chapter.

6.4.1.3 Flanged Construction

In components and elements consisting of a web and flange that act integrally, the combined stiffness and strength for flexural and axial loading shall be calculated considering a width of effective flange on each side of the web equal to the smaller of (1) the provided flange width, (2) eight times the flange thickness, (3) half the distance to the next web, and (4) one-fifth of the span for beams or one-half the total height for walls. When the flange is in compression, both the concrete and reinforcement within the effective width shall be considered effective in resisting flexure and axial load. When the flange is in tension, longitudinal reinforcement within the effective width shall be considered fully effective for resisting flexure and axial loads, provided that proper splice lengths in the reinforcement can be verified. The portion of the flange extending beyond the width of the web shall be assumed ineffective in resisting shear.

6.4.2 Design Strengths and Deformabilities

6.4.2.1 General

Actions in a structure shall be classified as being either deformation-controlled or force-controlled, as defined in Chapter 3. General procedures for calculating design strengths for deformation-controlled and force-controlled actions shall be according to Sections 6.4.2.2 and 6.4.2.3.

Components shall be classified as having low, moderate, or high ductility demands according to Section 6.4.2.4.

Where strength and deformation capacities are derived from test data, the tests shall be representative of proportions, details, and stress levels for the component. General requirements for testing are specified in Section 2.13.1.

Strengths and deformation capacities given in this chapter are for earthquake loadings involving three fully reversed deformation cycles to the design deformation levels, in addition to similar cycles to lesser deformation levels. In some cases—including some short-period buildings, and buildings subjected to a long-duration design earthquake—a building may be expected to be subjected to more numerous cycles to the design deformation levels. The increased number of cycles may lead to reductions in resistance and deformation capacity. The effects on strength and deformation capacity of more numerous deformation cycles should be considered in design. Large earthquakes will cause more numerous cycles.

6.4.2.2 Deformation-Controlled Actions

Deformation-controlled actions are defined in Section 3.2.2.4. Strengths used in design for deformation-controlled actions generally are denoted Q_{CE} and shall be taken as equal to expected strengths obtained experimentally or calculated using accepted mechanics principles. Expected strength is defined as the mean maximum resistance expected over the range of deformations to which the component is likely to be subjected. When calculations are used to define mean expected strength, expected material strength—including strain hardening—is to be taken into account. The tensile stress in yielding longitudinal reinforcement shall be assumed to be at least 1.25 times the nominal yield stress. Procedures specified in ACI 318 may be used to calculate strengths used in design, except that the strength reduction factor, ϕ, shall be taken as equal to unity, and other procedures specified in these *Guidelines* shall govern where applicable.

6.4.2.3 Force-Controlled Actions

Force-controlled actions are defined in Chapter 3. Strengths used in design for force-controlled actions generally are denoted Q_{CL} and shall be taken as equal to lower bound strengths obtained experimentally or calculated using established mechanics principles. Lower bound strength is defined generally as the lower five percentile of strengths expected. Where the strength degrades with continued cycling or increased lateral deformations, the lower bound strength is defined as the expected minimum value within the range of deformations and loading cycles to which the component is likely to be subjected. When calculations are used to define lower bound strengths, lower bound estimates of material properties are to be assumed. Procedures specified in ACI 318 may be used to calculate strengths used in design, except other procedures specified in the *Guidelines* shall govern where applicable (see Section 6.3.2.5).

6.4.2.4 Component Ductility Demand Classification

Some strength calculation procedures in this chapter require definition of component ductility demand classification. For this purpose, components shall also be classified as having low, moderate, or high ductility demands, based on the maximum value of the demand capacity ratio (DCR; see Section 2.9.1) from the linear procedures of Chapter 3, or the calculated displacement ductility from the nonlinear procedures of Chapter 3. Table 6-5 defines the relation.

Table 6-5 Component Ductility Demand Classification

Maximum value of DCR or displacement ductility	Descriptor
< 2	Low Ductility Demand
2 to 4	Moderate Ductility Demand
> 4	High Ductility Demand

6.4.3 Flexure and Axial Loads

Flexural strength and deformability of members with and without axial loads shall be calculated according to accepted procedures. Strengths and deformabilities of components with monolithic flanges shall be calculated considering concrete and developed longitudinal reinforcement within the effective flange width defined in Section 6.4.1.3. Strengths and deformabilities shall be determined considering available development of longitudinal reinforcement.

Without confining transverse reinforcement, maximum usable strain at extreme concrete compression fiber shall not exceed 0.002 for components in nearly pure compression and 0.005 for other components. Larger strains are permitted where transverse reinforcement provides confinement. Maximum allowable compression strains for confined concrete shall be based on experimental evidence and shall consider limitations posed by fracture of transverse reinforcement, buckling of longitudinal reinforcement, and degradation of component resistance at large deformation levels. Maximum compression strain shall not exceed 0.02, and maximum longitudinal reinforcement tension strain shall not exceed 0.05.

Where longitudinal reinforcement has embedment or development length into adjacent components that is insufficient for development of reinforcement strength—as in beams with bottom bars embedded a short distance into beam-column joints—flexural strength shall be calculated based on limiting stress capacity of the embedded bar as defined in Section 6.4.5.

Where flexural deformation capacities are calculated from basic mechanics principles, reduction in deformation capacity due to applied shear shall be taken into consideration.

6.4.4 Shear and Torsion

Strengths in shear and torsion shall be calculated according to ACI 318 (ACI, 1995), except as noted below and in Sections 6.5 and 6.9.

Within yielding regions of components with moderate or high ductility demands, shear and torsion strength shall be calculated according to accepted procedures for ductile components (for example, the provisions of Chapter 21 of *ACI 318-95*). Within yielding regions of components with low ductility demands, and outside yielding regions, shear strength may be calculated using accepted procedures normally used for elastic response (for example, the provisions of Chapter 11 of ACI 318).

Within yielding regions of components with moderate or high ductility demands, transverse reinforcement shall be assumed ineffective in resisting shear or torsion where: (1) longitudinal spacing of transverse reinforcement exceeds half the component effective depth measured in the direction of shear, or (2) perimeter hoops are either lap spliced or have hooks that are not adequately anchored in the concrete core.

Within yielding regions of components with low ductility demands, and outside yielding regions, transverse reinforcement shall be assumed ineffective in resisting shear or torsion where the longitudinal spacing of transverse reinforcement exceeds the component effective depth measured in the direction of shear.

Shear friction strength shall be calculated according to *ACI 318-95*, taking into consideration the expected axial load due to gravity and earthquake effects. Where rehabilitation involves addition of concrete requiring overhead work with dry-pack, the shear friction coefficient μ shall be taken as equal to 70% of the value specified by *ACI 318-95*.

6.4.5 Development and Splices of Reinforcement

Development strength of straight bars, hooked bars, and lap splices shall be calculated according to the general provisions of *ACI 318-95*, with the following modifications:

Within yielding regions of components with moderate or high ductility demands, details and strength provisions for new straight developed bars, hooked bars, and lap spliced bars shall be according to Chapter 21 of *ACI 318-95*. Within yielding regions of components with low ductility demands, and outside yielding regions, details and strength provisions for new construction shall be according to Chapter 12 of *ACI 318-95*, except requirements and strength provisions for lap splices may be taken as equal to those for straight development of bars in tension without consideration of lap splice classifications.

Where existing development, hook, and lap splice length and detailing requirements are not according to the requirements of the preceding paragraph, maximum stress capacity of reinforcement shall be calculated according to Equation 6-1.

$$f_s = \frac{l_b}{l_d} f_y \qquad (6\text{-}1)$$

where f_s = bar stress capacity for the development, hook, or lap splice length l_b provided; l_d = length required by Chapter 12 or Chapter 21 (as appropriate) of *ACI 318-95* for development, hook, or lap splice length, except splices may be assumed to be equivalent to straight bar development in tension; and f_y = yield strength of reinforcement. Where transverse reinforcement is distributed along the development length with spacing not exceeding one-third of the effective depth, the developed reinforcement may be assumed to retain the calculated stress capacity to large ductility levels. For larger spacings of transverse reinforcement, the developed stress shall be assumed to degrade from f_s to $0.2f_s$ at ductility demand or DCR equal to 2.0.

Strength of straight, discontinuous bars embedded in concrete sections (including beam-column joints) with clear cover over the embedded bar not less than $3d_b$ may be calculated according to Equation 6-2.

$$f_s = \frac{2500}{d_b} l_e \le f_y \qquad (6\text{-}2)$$

where f_s = maximum stress (in psi) that can be developed in an embedded bar having embedment length l_e (in inches), d_b = diameter of embedded bar (in inches), and f_y = bar yield stress (in psi). When the expected stress equals or exceeds f_s as calculated above, and f_s is less than f_y, the developed stress shall be assumed to degrade from f_s to $0.2f_s$ at ductility demand or DCR equal to 2.0. In beams with short bottom bar embedments into beam-column joints, flexural strength shall be calculated considering the stress limitation of Equation 6-2, and modeling parameters and acceptance criteria shall be according to Section 6.5.2.

Doweled bars added in seismic rehabilitation may be assumed to develop yield stress when all the following are satisfied: (1) drilled holes for dowel bars are cleaned with a stiff brush that extends the length of the hole; (2) embedment length l_e is not less than $10d_b$; and (3) minimum spacing of dowel bars is not less than $4l_e$, and minimum edge distance is not less than $2l_e$. Other design values for dowel bars shall be verified by test data. Field samples shall be obtained to ensure design strengths are developed per Section 6.3.3.

6.4.6 Connections to Existing Concrete

Connections used to connect two or more components may be classified according to their anchoring systems as cast-in-place systems or as post-installed systems.

6.4.6.1 Cast-In-Place Systems

The capacity of the connection should be not less than 1.25 times the smaller of (1) the force corresponding to development of the minimum probable strength of the two interconnected components, and (2) the component actions at the connection. Shear forces, tension forces, bending moments, and prying actions shall be considered. Design values for connection anchorages shall be ultimate values, and shall be taken as suggested in ACI Report 355.1R-91, or as specified in the latest version of the locally adopted strength design building code.

The capacity of anchors placed in areas where cracking is expected shall be reduced by a factor of 0.5.

6.4.6.2 Post-Installed Systems

The capacity should be calculated according to Section 6.4.6.1. See the *Commentary* for exceptions.

6.4.6.3 Quality Control

See *Commentary* for this section.

6.5 Concrete Moment Frames

6.5.1 Types of Concrete Moment Frames

Concrete moment frames are those elements composed primarily of horizontal framing components (beams and/or slabs) and vertical framing components (columns) that develop lateral load resistance through bending of horizontal and vertical framing components. These elements may act alone to resist lateral loads, or they may act in conjunction with shear walls, braced frames, or other elements to form a dual system.

The provisions in Section 6.5 are applicable to frames that are cast monolithically, including monolithic concrete frames rehabilitated or created by the addition of new material. Frames covered under this section include reinforced concrete beam-column moment frames, prestressed concrete beam-column moment frames, and slab-column moment frames. Sections 6.6, 6.7, and 6.10 apply to precast concrete frames, infilled concrete frames, and concrete braced frames, respectively.

6.5.1.1 Reinforced Concrete Beam-Column Moment Frames

Reinforced concrete beam-column moment frames are those frames that satisfy the following conditions:

1. Framing components are beams (with or without slabs) and columns.
2. Beams and columns are of monolithic construction that provides for moment transfer between beams and columns.
3. Primary reinforcement in components contributing to lateral load resistance is nonprestressed.

The frames include Special Moment Frames, Intermediate Moment Frames, and Ordinary Moment Frames as defined in the 1994 *NEHRP Recommended Provisions* (BSSC, 1995), as well as frames not satisfying the requirements of these *Provisions*. This classification includes existing construction, new construction, and existing construction that has been rehabilitated.

6.5.1.2 Post-Tensioned Concrete Beam-Column Moment Frames

Post-tensioned concrete beam-column moment frames are those frames that satisfy the following conditions:

1. Framing components are beams (with or without slabs) and columns.
2. Beams and columns are of monolithic construction that provides for moment transfer between beams and columns.
3. Primary reinforcement in beams contributing to lateral load resistance includes post-tensioned reinforcement with or without nonprestressed reinforcement.

This classification includes existing construction, new construction, and existing construction that has been rehabilitated.

6.5.1.3 Slab-Column Moment Frames

Slab-column moment frames are those frames that satisfy the following conditions:

1. Framing components are slabs (with or without beams in the transverse direction) and columns.

2. Slabs and columns are of monolithic construction that provides for moment transfer between slabs and columns.

3. Primary reinforcement in slabs contributing to lateral load resistance includes nonprestressed reinforcement, prestressed reinforcement, or both.

The slab-column frame may or may not have been intended in the original design to be part of the lateral-load-resisting system. This classification includes existing construction, new construction, and existing construction that has been rehabilitated.

6.5.2 Reinforced Concrete Beam-Column Moment Frames

6.5.2.1 General Considerations

The analysis model for a beam-column frame element shall represent strength, stiffness, and deformation capacity of beams, columns, beam-column joints, and other components that may be part of the frame, including connections with other elements. Potential failure in flexure, shear, and reinforcement development at any section along the component length shall be considered. Interaction with other elements, including nonstructural elements and components, shall be included.

The analytical model generally can represent a beam-column frame using line elements with properties concentrated at component centerlines. Where beam and column centerlines do not coincide, the effects on framing shall be considered. Where minor eccentricities occur (i.e., the centerline of the narrower component falls within the middle third of the adjacent framing component measured transverse to the framing direction), the effect of the eccentricity can be ignored. Where larger eccentricities occur, the effect shall be represented either by reductions in effective stiffnesses, strengths, and deformation capacities, or by direct modeling of the eccentricity.

The beam-column joint in monolithic construction generally shall be represented as a stiff or rigid zone having horizontal dimensions equal to the column cross-sectional dimensions and vertical dimension equal to the beam depth, except that a wider joint may be assumed where the beam is wider than the column and where justified by experimental evidence. The model of the connection between the columns and foundation shall be selected based on the details of the column-foundation connection and rigidity of the foundation-soil system.

Action of the slab as a diaphragm interconnecting vertical elements shall be represented. Action of the slab as a composite beam flange is to be considered in developing stiffness, strength, and deformation capacities of the beam component model, according to Section 6.4.1.3.

Inelastic deformations in primary components shall be restricted to flexure in beams (plus slabs, if present) and columns. Other inelastic deformations are permitted in secondary components. Acceptance criteria are provided in Section 6.5.2.4.

6.5.2.2 Stiffness for Analysis

A. Linear Static and Dynamic Procedures

Beams shall be modeled considering flexural and shear stiffnesses, including in monolithic construction the effect of the slab acting as a flange. Columns shall be modeled considering flexural, shear, and axial stiffnesses. Joints shall be modeled as stiff components, and may in most cases be considered rigid. Effective stiffnesses shall be according to Section 6.4.1.2.

B. Nonlinear Static Procedure

Nonlinear load-deformation relations shall follow the general guidelines of Section 6.4.1.2.

Beams and columns may be modeled using concentrated plastic hinge models, distributed plastic hinge models, or other models whose behavior has been demonstrated to adequately represent important characteristics of reinforced concrete beam and column components subjected to lateral loading. The model shall be capable of representing inelastic response along the component length, except where it is shown by equilibrium that yielding is restricted to the component ends. Where nonlinear response is expected in a mode other than flexure, the model shall be established to represent these effects.

Monotonic load-deformation relations shall be according to the generalized relation shown in Figure 6-1, except that different relations are permitted where verified by experiments. In that figure, point B corresponds to significant yielding, C corresponds to the point where significant lateral load resistance can be assumed to be lost, and E corresponds to the point where gravity load resistance can be assumed to be lost.

The overall load-deformation relation shall be established so that the maximum resistance is consistent with the design strength specifications of Sections 6.4.2 and 6.5.2.3.

For beams and columns, the generalized deformation in Figure 6-1 may be either the chord rotation or the plastic hinge rotation. For beam-column joints, an acceptable measure of the generalized deformation is shear strain. Values of the generalized deformation at points B, C, and D may be derived from experiments or rational analyses, and shall take into account the interactions between flexure, axial load, and shear. Alternately, where the generalized deformation is taken as rotation in the flexural plastic hinge zone in beams and columns, the plastic hinge rotation capacities shall be as defined by Tables 6-6 and 6-7. Where the generalized deformation is shear distortion of the beam-column joint, shear angle capacities shall be as defined by Table 6-8.

C. Nonlinear Dynamic Procedure

For the NDP, the complete hysteresis behavior of each component shall be modeled using properties verified by experimental evidence. The relation of Figure 6-1 may be taken to represent the envelope relation for the analysis. Unloading and reloading properties shall represent significant stiffness and strength degradation characteristics.

6.5.2.3 Design Strengths

Component strengths shall be computed according to the general requirements of Section 6.4.2, as modified in this section.

The maximum component strength shall be determined considering potential failure in flexure, axial load, shear, torsion, development, and other actions at all points along the length of the component under the actions of design gravity and lateral load combinations.

For columns, the contribution of concrete to shear strength, V_c, may be calculated according to Equation 6-3.

$$V_c = 3.5\lambda\left(k + \frac{N_u}{2000A_g}\right)\sqrt{f'_c}\, b_w d \qquad (6\text{-}3)$$

in which k = 1.0 in regions of low ductility demand and 0 in regions of moderate and high ductility demand, λ = 0.75 for lightweight aggregate concrete and 1.0 for normal weight aggregate concrete, and N_u = axial compression force in pounds (= 0 for tension force). All units are expressed in pounds and inches. Where axial force is calculated from the linear procedures of Chapter 3, compressive axial load for use in Equation 6-3 should be taken as equal to the value calculated considering design gravity load only, and tensile axial load should be taken as equal to the value calculated from the analysis considering design load combinations, including gravity and earthquake loading according to Section 3.2.8.

For columns satisfying the detailing and proportioning requirements of Chapter 21 of ACI 318, the shear strength equations of ACI 318 may be used.

For beam-column joints, the nominal cross-sectional area, A_j, shall be defined by a joint depth equal to the column dimension in the direction of framing and a joint width equal to the smallest of (1) the column width, (2) the beam width plus the joint depth, and (3) twice the smaller perpendicular distance from the longitudinal axis of the beam to the column side. Design forces shall be calculated based on development of flexural plastic hinges in adjacent framing members, including effective slab width, but need not exceed values calculated from design gravity and earthquake load combinations. Nominal joint shear strength V_n shall be calculated according to the general procedures of ACI 318, modified as described below.

$$Q_{CL} = V_n = \lambda\gamma\sqrt{f'_c}\,A_j,\ psi \qquad (6\text{-}4)$$

in which λ = 0.75 for lightweight aggregate concrete and 1.0 for normal weight aggregate concrete, A_j is the effective horizontal joint area with dimension as defined above, and γ is as defined in Table 6-9.

6.5.2.4 Acceptance Criteria

A. Linear Static and Dynamic Procedures

All actions shall be classified as being either deformation-controlled or force-controlled, as defined in Chapter 3. In primary components, deformation-controlled actions shall be restricted to flexure in beams (with or without slab) and columns. In secondary components, deformation-controlled actions shall be restricted to flexure in beams (with or without slab), plus restricted actions in shear and reinforcement development, as identified in Tables 6-10 through 6-12.

Table 6-6 Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Reinforced Concrete Beams

Conditions			Modeling Parameters[3]			Acceptance Criteria[3]				
						Plastic Rotation Angle, radians				
						Component Type				
						Primary			Secondary	
			Plastic Rotation Angle, radians		Residual Strength Ratio	Performance Level				
			a	b	c	IO	LS	CP	LS	CP
i. Beams controlled by flexure[1]										
$\frac{\rho - \rho'}{\rho_{bal}}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f'_c}}$								
≤ 0.0	C	≤ 3	0.025	0.05	0.2	0.005	0.02	0.025	0.02	0.05
≤ 0.0	C	≥ 6	0.02	0.04	0.2	0.005	0.01	0.02	0.02	0.04
≥ 0.5	C	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≥ 0.5	C	≥ 6	0.015	0.02	0.2	0.005	0.005	0.015	0.015	0.02
≤ 0.0	NC	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≤ 0.0	NC	≥ 6	0.01	0.015	0.2	0.0	0.005	0.01	0.01	0.015
≥ 0.5	NC	≤ 3	0.01	0.015	0.2	0.005	0.01	0.01	0.01	0.015
≥ 0.5	NC	≥ 6	0.005	0.01	0.2	0.0	0.005	0.005	0.005	0.01
ii. Beams controlled by shear[1]										
Stirrup spacing ≤ d/2			0.0	0.02	0.2	0.0	0.0	0.0	0.01	0.02
Stirrup spacing > d/2			0.0	0.01	0.2	0.0	0.0	0.0	0.005	0.01
iii. Beams controlled by inadequate development or splicing along the span[1]										
Stirrup spacing ≤ d/2			0.0	0.02	0.0	0.0	0.0	0.0	0.01	0.02
Stirrup spacing > d/2			0.0	0.01	0.0	0.0	0.0	0.0	0.005	0.01
iv. Beams controlled by inadequate embedment into beam-column joint[1]										
			0.015	0.03	0.2	0.01	0.01	0.015	0.02	0.03

1. When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
2. Under the heading "Transverse Reinforcement," "C" and "NC" are abbreviations for conforming and nonconforming details, respectively. A component is conforming if, within the flexural plastic region, closed stirrups are spaced at ≤ d/3, and if, for components of moderate and high ductility demand, the strength provided by the stirrups (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.
3. Linear interpolation between values listed in the table is permitted.

All other actions shall be defined as being force-controlled actions.

Design actions on components shall be determined as prescribed in Chapter 3. Where the calculated DCR values exceed unity, the following actions preferably shall be determined using limit analysis principles as prescribed in Chapter 3: (1) moments, shears, torsions, and development and splice actions corresponding to development of component strength in beams and

Table 6-7 Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Reinforced Concrete Columns

			Modeling Parameters[4]			Acceptance Criteria[4]				
						Plastic Rotation Angle, radians				
						Component Type				
						Primary			Secondary	
			Plastic Rotation Angle, radians		Residual Strength Ratio	Performance Level				
Conditions			**a**	**b**	**c**	**IO**	**LS**	**CP**	**LS**	**CP**
i. Columns controlled by flexure[1]										
$\frac{P}{A_g f'_c}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f'_c}}$								
≤ 0.1	C	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.015	0.03
≤ 0.1	C	≥ 6	0.015	0.025	0.2	0.005	0.01	0.015	0.01	0.025
≥ 0.4	C	≤ 3	0.015	0.025	0.2	0.0	0.005	0.015	0.010	0.025
≥ 0.4	C	≥ 6	0.01	0.015	0.2	0.0	0.005	0.01	0.01	0.015
≤ 0.1	NC	≤ 3	0.01	0.015	0.2	0.005	0.005	0.01	0.005	0.015
≤ 0.1	NC	≥ 6	0.005	0.005	–	0.005	0.005	0.005	0.005	0.005
≥ 0.4	NC	≤ 3	0.005	0.005	–	0.0	0.0	0.005	0.0	0.005
≥ 0.4	NC	≥ 6	0.0	0.0	–	0.0	0.0	0.0	0.0	0.0
ii. Columns controlled by shear[1,3]										
Hoop spacing ≤ d/2, or $\frac{P}{A_g f'_c} \leq 0.1$			0.0	0.015	0.2	0.0	0.0	0.0	0.01	0.015
Other cases			0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
iii. Columns controlled by inadequate development or splicing along the clear height[1,3]										
Hoop spacing ≤ d/2			0.01	0.02	0.4	1	1	1	0.01	0.02
Hoop spacing > d/2			0.0	0.01	0.2	1	1	1	0.005	0.01
iv. Columns with axial loads exceeding 0.70P_o[1,3]										
Conforming reinforcement over the entire length			0.015	0.025	0.02	0.0	0.005	0.001	0.01	0.02
All other cases			0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

1. When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
2. Under the heading "Transverse Reinforcement," "C" and "NC" are abbreviations for conforming and nonconforming details, respectively. A component is conforming if, within the flexural plastic hinge region, closed hoops are spaced at ≤ d/3, and if, for components of moderate and high ductility demand, the strength provided by the stirrups (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.
3. To qualify, hoops must not be lap spliced in the cover concrete, and hoops must have hooks embedded in the core or other details to ensure that hoops will be adequately anchored following spalling of cover concrete.
4. Linear interpolation between values listed in the table is permitted.

Table 6-8 Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Reinforced Concrete Beam-Column Joints

Conditions			Modeling Parameters[4]			Acceptance Criteria[4]				
						Plastic Rotation Angle, radians				
						Component Type				
						Primary			Secondary	
			Shear Angle, radians		Residual Strength Ratio	Performance Level				
			d	e	c	IO	LS	CP	LS	CP
i. Interior joints										
$\frac{P}{A_g f'_c}$ [2]	Trans. Reinf.[1]	$\frac{V}{V_n}$ [3]								
≤ 0.1	C	≤ 1.2	0.015	0.03	0.2	0.0	0.0	0.0	0.02	0.03
≤ 0.1	C	≥ 1.5	0.015	0.03	0.2	0.0	0.0	0.0	0.015	0.02
≥ 0.4	C	≤ 1.2	0.015	0.025	0.2	0.0	0.0	0.0	0.015	0.025
≥ 0.4	C	≥ 1.5	0.015	0.02	0.2	0.0	0.0	0.0	0.015	0.02
≤ 0.1	NC	≤ 1.2	0.005	0.02	0.2	0.0	0.0	0.0	0.015	0.02
≤ 0.1	NC	≥ 1.5	0.005	0.015	0.2	0.0	0.0	0.0	0.01	0.015
≥ 0.4	NC	≤ 1.2	0.005	0.015	0.2	0.0	0.0	0.0	0.01	0.015
≥ 0.4	NC	≥ 1.5	0.005	0.015	0.2	0.0	0.0	0.0	0.01	0.015
ii. Other joints										
$\frac{P}{A_g f'_c}$ [2]	Trans. Reinf.[1]	$\frac{V}{V_n}$ [3]								
≤ 0.1	C	≤ 1.2	0.01	0.02	0.2	0.0	0.0	0.0	0.015	0.02
≤ 0.1	C	≥ 1.5	0.01	0.015	0.2	0.0	0.0	0.0	0.01	0.015
≥ 0.4	C	≤ 1.2	0.01	0.02	0.2	0.0	0.0	0.0	0.015	0.02
≥ 0.4	C	≥ 1.5	0.01	0.015	0.2	0.0	0.0	0.0	0.01	0.015
≤ 0.1	NC	≤ 1.2	0.005	0.01	0.2	0.0	0.0	0.0	0.005	0.01
≤ 0.1	NC	≥ 1.5	0.005	0.01	0.2	0.0	0.0	0.0	0.005	0.01
≥ 0.4	NC	≤ 1.2	0.0	0.0	–	0.0	0.0	0.0	0.0	0.0
≥ 0.4	NC	≥ 1.5	0.0	0.0	–	0.0	0.0	0.0	0.0	0.0

1. Under the heading "Transverse Reinforcement," "C" and "NC" are abbreviations for conforming and nonconforming details, respectively. A joint is conforming if closed hoops are spaced at ≤ $h_c/3$ within the joint. Otherwise, the component is considered nonconforming. Also, to qualify as conforming details under ii, hoops must not be lap spliced in the cover concrete, and must have hooks embedded in the core or other details to ensure that hoops will be adequately anchored following spalling of cover concrete.
2. This is the ratio of the design axial force on the column above the joint to the product of the gross cross-sectional area of the joint and the concrete compressive strength. The design axial force is to be calculated using limit analysis procedures, as described in Chapter 3.
3. This is the ratio of the design shear force to the shear strength for the joint. The design shear force is to be calculated according to Section 6.5.2.3.
4. Linear interpolation between values listed in the table is permitted.

Table 6-9 Values of γ for Joint Strength Calculation

ρ''	Value of γ				
	Interior joint with transverse beams	**Interior joint without transverse beams**	**Exterior joint with transverse beams**	**Exterior joint without transverse beams**	**Knee joint**
<0.003	12	10	8	6	4
≥0.003	20	15	15	12	8

ρ'' = volumetric ratio of horizontal confinement reinforcement in the joint; knee joint = self-descriptive—with transverse beams or not.

columns; (2) joint shears corresponding to development of strength in adjacent beams and/or columns; and (3) axial load in columns and joints, considering likely plastic action in components above the level in question.

Design actions shall be compared with design strengths to determine which components develop their design strengths. Those components that satisfy Equations 3-18 and 3-19 may be assumed to satisfy the performance criteria for those components. Components that reach their design strengths shall be further evaluated according to Section 6.5.2.4A to determine performance acceptability.

Where the average DCR of columns at a level exceeds the average value of beams at the same level, and exceeds the greater of 1.0 and $m/2$ for columns, the element is defined as a weak story element. For weak story elements, one of the following shall be satisfied.

1. The check of average DCR values at the level is repeated, considering all elements in the building system. If the average of the DCR values for vertical components exceeds the average value for horizontal components at the level, and exceeds 2.0, the structure shall be reanalyzed using a nonlinear procedure, or the structure shall be rehabilitated to remove this deficiency.

2. The structure shall be reanalyzed using either the NSP or the NDP of Chapter 3.

3. The structure shall be rehabilitated to remove this deficiency.

Calculated component actions shall satisfy the requirements of Chapter 3. Tables 6-10 through 6-12 present m values for use in Equation 3-18. Alternative approaches or values are permitted where justified by experimental evidence and analysis.

B. Nonlinear Static and Dynamic Procedures

Inelastic response shall be restricted to those components and actions listed in Tables 6-6 through 6-8, except where it is demonstrated that other inelastic action can be tolerated considering the selected Performance Levels.

Calculated component actions shall satisfy the requirements of Chapter 3. Maximum permissible inelastic deformations are listed in Tables 6-6 through 6-8. Where inelastic action is indicated for a component or action not listed in these tables, the performance shall be deemed unacceptable. Alternative approaches or values are permitted where justified by experimental evidence and analysis.

6.5.2.5 Rehabilitation Measures

Rehabilitation measures include the following general approaches, plus other approaches based on rational procedures.

- **Jacketing existing beams, columns, or joints with new reinforced concrete, steel, or fiber wrap overlays**. The new materials shall be designed and constructed to act compositely with the existing concrete. Where reinforced concrete jackets are used, the design shall provide detailing to enhance ductility. Component strength shall be taken to not exceed any limiting strength of connections with adjacent components. Jackets designed to provide increased connection strength and improved continuity between adjacent components are permitted.

Table 6-10 Numerical Acceptance Criteria for Linear Procedures—Reinforced Concrete Beams

Conditions			*m* factors[3]				
			Component Type				
			Primary			Secondary	
			Performance Level				
			IO	LS	CP	LS	CP
i. Beams controlled by flexure[1]							
$\frac{\rho - \rho'}{\rho_{bal}}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f'_c}}$					
≤ 0.0	C	≤ 3	2	6	7	6	10
≤ 0.0	C	≥ 6	2	3	4	3	5
≥ 0.5	C	≤ 3	2	3	4	3	5
≥ 0.5	C	≥ 6	2	2	3	2	4
≤ 0.0	NC	≤ 3	2	3	4	3	5
≤ 0.0	NC	≥ 6	1	2	3	2	4
≥ 0.5	NC	≤ 3	2	3	3	3	4
≥ 0.5	NC	≥ 6	1	2	2	2	3
ii. Beams controlled by shear[1]							
Stirrup spacing ≤ $d/2$			–	–	–	3	4
Stirrup spacing > $d/2$			–	–	–	2	3
iii. Beams controlled by inadequate development or splicing along the span[1]							
Stirrup spacing ≤ $d/2$			–	–	–	3	4
Stirrup spacing > $d/2$			–	–	–	2	3
iv. Beams controlled by inadequate embedment into beam-column joint[1]							
			2	2	3	3	4

1. When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
2. Under the heading "Transverse Reinforcement," "C" and "NC" are abbreviations for conforming and nonconforming details, respectively. A component is conforming if, within the flexural plastic region, closed stirrups are spaced at ≤ $d/3$, and if, for components of moderate and high ductility demand, the strength provided by the stirrups (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.
3. Linear interpolation between values listed in the table is permitted.

- **Post-tensioning existing beams, columns, or joints using external post-tensioned reinforcement.** Post-tensioned reinforcement shall be unbonded within a distance equal to twice the effective depth from sections where inelastic action is expected. Anchorages shall be located away from regions where inelastic action is anticipated, and shall be designed considering possible force variations due to earthquake loading.
- **Modification of the element by selective material removal from the existing element.** Examples include: (1) where nonstructural elements or components interfere with the frame, removing or separating the nonstructural elements or components to eliminate the interference; (2) weakening, usually by removal of concrete or severing of longitudinal reinforcement, to change response mode from a nonductile mode to a more ductile mode (e.g.,

Table 6-11 Numerical Acceptance Criteria for Linear Procedures—Reinforced Concrete Columns

Conditions			*m* factors[4]				
			Component Type				
			Primary			**Secondary**	
			Performance Level				
			IO	**LS**	**CP**	**LS**	**CP**
i. Columns controlled by flexure[1]							
$\frac{P}{A_g f'_c}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f'_c}}$					
≤ 0.1	C	≤ 3	2	3	4	3	4
≤ 0.1	C	≥ 6	2	3	3	3	3
≥ 0.4	C	≤ 3	1	2	2	2	2
≥ 0.4	C	≥ 6	1	1	2	1	2
≤ 0.1	NC	≤ 3	2	2	3	2	3
≤ 0.1	NC	≥ 6	2	2	2	2	2
≥ 0.4	NC	≤ 3	1	1	2	1	2
≥ 0.4	NC	≥ 6	1	1	1	1	1
ii. Columns controlled by shear[1,3]							
Hoop spacing ≤ $d/2$, or $\frac{P}{A_g f'_c} \leq 0.1$			–	–	–	2	3
Other cases			–	–	–	1	1
iii. Columns controlled by inadequate development or splicing along the clear height[1,3]							
Hoop spacing ≤ $d/2$			–	–	–	3	4
Hoop spacing > $d/2$			–	–	–	2	3
iv. Columns with axial loads exceeding $0.70P_o$[1,3]							
Conforming reinforcement over the entire length			1	1	2	2	2
All other cases			–	–	–	1	1

1. When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
2. Under the heading "Transverse Reinforcement," "C" and "NC" are abbreviations for conforming and nonconforming details, respectively. A component is conforming if, within the flexural plastic hinge region, closed hoops are spaced at ≤ $d/3$, and if, for components of moderate and high ductility demand, the strength provided by the stirrups (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.
3. To qualify, hoops must not be lap spliced in the cover concrete, and must have hooks embedded in the core or other details to ensure that hoops will be adequately anchored following spalling of cover concrete.
4. Linear interpolation between values listed in the table is permitted.

weakening of beams to promote formation of a strong-column, weak-beam system); and (3) segmenting walls to change stiffness and strength.

Table 6-12 Numerical Acceptance Criteria for Linear Procedures—Reinforced Concrete Beam-Column Joints

Conditions			*m* factors[4]				
			Component Type				
			Primary[5]			Secondary	
			Performance Level				
			IO	LS	CP	LS	CP
i. Interior joints							
$\frac{P}{A_g f'_c}$ [2]	Trans. Reinf.[1]	$\frac{V}{V_n}$ [3]					
≤ 0.1	C	≤ 1.2	–	–	–	3	4
≤ 0.1	C	≥ 1.5	–	–	–	2	3
≥ 0.4	C	≤ 1.2	–	–	–	3	4
≥ 0.4	C	≥ 1.5	–	–	–	2	3
≤ 0.1	NC	≤ 1.2	–	–	–	2	3
≤ 0.1	NC	≥ 1.5	–	–	–	2	3
≥ 0.4	NC	≤ 1.2	–	–	–	2	3
≥ 0.4	NC	≥ 1.5	–	–	–	2	3
ii. Other joints							
$\frac{P}{A_g f'_c}$ [2]	Trans. Reinf.[1]	$\frac{V}{V_n}$ [3]					
≤ 0.1	C	≤ 1.2	–	–	–	3	4
≤ 0.1	C	≥ 1.5	–	–	–	2	3
≥ 0.4	C	≤ 1.2	–	–	–	3	4
≥ 0.4	C	≥ 1.5	–	–	–	2	3
≤ 0.1	NC	≤ 1.2	–	–	–	2	3
≤ 0.1	NC	≥ 1.5	–	–	–	2	3
≥ 0.4	NC	≤ 1.2	–	–	–	1	1
≥ 0.4	NC	≥ 1.5	–	–	–	1	1

1. Under the heading “Transverse Reinforcement,” “C” and “NC” are abbreviations for conforming and nonconforming details, respectively. A joint is conforming if closed hoops are spaced at ≤ $h_c/3$ within the joint. Otherwise, the component is considered nonconforming. Also, to qualify as conforming details under ii, hoops must not be lap spliced in the cover concrete, and must have hooks embedded in the core or other details to ensure that hoops will be adequately anchored following spalling of cover concrete.
2. This is the ratio of the design axial force on the column above the joint to the product of the gross cross-sectional area of the joint and the concrete compressive strength. The design axial force is to be calculated using limit analysis procedures as described in Chapter 3.
3. This is the ratio of the design shear force to the shear strength for the joint. The design shear force is to be calculated according to Section 6.5.2.3.
4. Linear interpolation between values listed in the table is permitted.
5. All interior joints are force-controlled, and no *m* factors apply.

- **Improvement of deficient existing reinforcement details.** This approach involves removal of cover concrete, modification of existing reinforcement details, and casting of new cover concrete. Concrete removal shall avoid unintended damage to core concrete and the bond between existing reinforcement and core concrete. New cover concrete shall be designed and constructed to achieve fully composite action with the existing materials.

- **Changing the building system to reduce the demands on the existing element.** Examples include addition of supplementary lateral-force-resisting elements such as walls or buttresses, seismic isolation, and mass reduction.

- **Changing the frame element to a shear wall, infilled frame, or braced frame element by addition of new material.** Connections between new and existing materials shall be designed to transfer the forces anticipated for the design load combinations. Where the existing concrete frame columns and beams act as boundary elements and collectors for the new shear wall or braced frame, these shall be checked for adequacy, considering strength, reinforcement development, and deformability. Diaphragms, including drag struts and collectors, shall be evaluated and, if necessary, rehabilitated to ensure a complete load path to the new shear wall or braced frame element.

Rehabilitated frames shall be evaluated according to the general principles and requirements of this chapter. The effects of rehabilitation on stiffness, strength, and deformability shall be taken into account in an analytical model of the rehabilitated structure. Connections required between existing and new elements shall satisfy the requirements of Section 6.4.6 and other requirements of the *Guidelines*. An existing frame rehabilitated according to procedures listed above shall satisfy the relevant specific requirements of Chapter 6.

6.5.3 Post-Tensioned Concrete Beam-Column Moment Frames

6.5.3.1 General Considerations

The analysis model for a post-tensioned concrete beam-column frame element shall be established following the guidelines established in Section 6.5.2.1 for reinforced concrete beam-column moment frames. In addition to potential failure modes described in Section 6.5.2.1, the analysis model shall consider potential failure of tendon anchorages.

The linear procedures and the NSP described in Chapter 3 apply directly to frames with post-tensioned beams in which the following conditions are satisfied:

1. The average prestress, f_{pc}, calculated for an area equal to the product of the shortest cross-sectional dimension and the perpendicular cross-sectional dimension of the beam, does not exceed the greater of 350 psi or f'_c /12 at locations of nonlinear action.

2. Prestressing tendons do not provide more than one-quarter of the strength for both positive moments and negative moments at the joint face.

3. Anchorages for tendons have been demonstrated to perform satisfactorily for seismic loadings. These anchorages must occur outside hinging areas or joints.

Alternative procedures are required where these conditions are not satisfied.

6.5.3.2 Stiffness for Analysis

A. Linear Static and Dynamic Procedures

Beams shall be modeled considering flexural and shear stiffnesses, including in monolithic and composite construction the effect of the slab acting as a flange. Columns shall be modeled considering flexural, shear, and axial stiffnesses. Joints shall be modeled as stiff components, and may in most cases be considered rigid. Effective stiffnesses shall be according to Section 6.4.1.2.

B. Nonlinear Static Procedure

Nonlinear load-deformation relations shall follow the general guidelines of Section 6.4.1.2 and the reinforced concrete frame guidelines of Section 6.5.2.2B.

Values of the generalized deformation at points *B*, *C*, and *D* in Figure 6-1 may be derived from experiments or rational analyses, and shall take into account the interactions between flexure, axial load, and shear. Alternately, where the generalized deformation is taken as rotation in the flexural plastic hinge zone, and where the three conditions of Section 6.5.3.1 are satisfied, beam plastic hinge rotation capacities may be as defined by Table 6-6. Columns and joints may be modeled as described in Section 6.5.2.2.

C. Nonlinear Dynamic Procedure

For the NDP, the complete hysteresis behavior of each component shall be modeled using properties verified by experimental evidence. The relation of Figure 6-1 may be taken to represent the envelope relation for the analysis. Unloading and reloading properties shall represent significant stiffness and strength degradation characteristics as influenced by prestressing.

6.5.3.3 Design Strengths

Component strengths shall be computed according to the general requirements of Section 6.4.2 and the additional requirements of Section 6.5.2.3. Effects of prestressing on strength shall be considered. For deformation-controlled actions, prestress shall be assumed to be effective for the purpose of determining the maximum actions that may be developed associated with nonlinear response of the frame. For force-controlled actions, the effects on strength of prestress loss shall also be considered as a design condition, where these losses are possible under design load combinations including inelastic deformation reversals.

6.5.3.4 Acceptance Criteria

Acceptance criteria shall follow the criteria for reinforced concrete beam-column frames, as specified in Section 6.5.2.4.

Tables 6-6, 6-7, 6-8, 6-10, 6-11, and 6-12 present acceptability values for use in the four procedures of Chapter 3. The values in these tables for beams apply only if the beams satisfy the three conditions of Section 6.5.3.1.

6.5.3.5 Rehabilitation Measures

Rehabilitation measures include the general approaches listed in Section 6.5.2.5, as well as other approaches based on rational procedures.

Rehabilitated frames shall be evaluated according to the general principles and requirements of this chapter. The effects of rehabilitation on stiffness, strength, and deformability shall be taken into account in an analytical model of the rehabilitated building. Connections required between existing and new elements shall satisfy the requirements of Section 6.4.6 and other requirements of the *Guidelines*.

6.5.4 Slab-Column Moment Frames

6.5.4.1 General Considerations

The analysis model for a slab-column frame element shall represent strength, stiffness, and deformation capacity of slabs, columns, slab-column connections, and other components that may be part of the frame. Potential failure in flexure, shear, shear-moment transfer, and reinforcement development at any section along the component length shall be considered. Interaction with other elements, including nonstructural elements and components, shall be included.

The analytical model can represent the slab-column frame, using line elements with properties concentrated at component centerlines, or a combination of line elements (to represent columns) and plate-bending elements (to represent the slab). Three approaches are specifically recognized.

- **Effective beam width model.** Columns and slabs are represented by frame elements that are rigidly interconnected at the slab-column joint.
- **Equivalent frame model.** Columns and slabs are represented by frame elements that are interconnected by connection springs.
- **Finite element model.** The columns are represented by frame elements and the slab is represented by plate-bending elements.

In any model, the effects of changes in cross section, including slab openings, shall be considered.

The model of the connection between the columns and foundation shall be selected based on the details of the column-foundation connection and rigidity of the foundation-soil system.

Action of the slab as a diaphragm interconnecting vertical elements shall be represented.

Inelastic deformations in primary components shall be restricted to flexure in slabs and columns, plus limited nonlinear response in slab-column connections. Other inelastic deformations are permitted in secondary components. Acceptance criteria are in Section 6.5.4.4.

6.5.4.2 Stiffness for Analysis

A. Linear Static and Dynamic Procedures

Slabs shall be modeled considering flexural, shear, and tension (in the slab adjacent to the column) stiffnesses. Columns shall be modeled considering flexural, shear, and axial stiffnesses. Joints shall be modeled as stiff components, and may in most cases be considered rigid. The effective stiffnesses of components shall be adjusted on the basis of experimental evidence to represent effective stiffnesses according to the general principles of Section 6.4.1.2.

B. Nonlinear Static Procedure

Nonlinear load-deformation relations shall follow the general guidelines of Section 6.4.1.2.

Slabs and columns may be modeled using concentrated plastic hinge models, distributed plastic hinge models, or other models whose behavior has been demonstrated to adequately represent important characteristics of reinforced concrete slab and column components subjected to lateral loading. The model shall be capable of representing inelastic response along the component length, except where it is shown by equilibrium that yielding is restricted to the component ends. Slab-column connections preferably will be modeled separately from the slab and column components, so that potential failure in shear and moment transfer can be identified. Where nonlinear response is expected in a mode other than flexure, the model shall be established to represent these effects.

Monotonic load-deformation relations shall be according to the generalized relation shown in Figure 6-1, with definitions according to Section 6.5.2.2B. The overall load-deformation relation shall be established so that the maximum resistance is consistent with the design strength specifications of Sections 6.4.2 and 6.5.4.3. Where the generalized deformation shown in Figure 6-1 is taken as the flexural plastic hinge rotation for the column, the plastic hinge rotation capacities shall be as defined by Table 6-7. Where the generalized deformation shown in Figure 6-1 is taken as the rotation of the slab-column connection, the plastic rotation capacities shall be as defined by Table 6-13.

C. Nonlinear Dynamic Procedure

The general approach shall be according to the specification of Section 6.5.2.2C.

6.5.4.3 Design Strengths

Component strengths shall be according to the general requirements of Section 6.4.2, as modified in this section.

The maximum component strength shall be determined considering potential failure in flexure, axial load, shear, torsion, development, and other actions at all points along the length of the component under the actions of design gravity and lateral load combinations. The strength of slab-column connections shall also be determined and incorporated in the analytical model.

The flexural strength of a slab to resist moment due to lateral deformations shall be calculated as $M_{nCS} - M_{gCS}$, where M_{nCS} is the design flexural strength of the column strip and M_{gCS} is the column strip moment due to gravity loads. M_{gCS} is to be calculated according to the procedures of *ACI 318-95* (ACI, 1995) for the design gravity load specified in Chapter 3.

For columns, the shear strength may be evaluated according to Section 6.5.2.3.

Shear and moment transfer strength of the slab-column connection shall be calculated considering the combined action of flexure, shear, and torsion acting in the slab at the connection with the column. An acceptable procedure is to calculate the shear and moment transfer strength as described below.

For interior connections without transverse beams, and for exterior connections with moment about an axis perpendicular to the slab edge, the shear and moment transfer strength may be taken as equal to the minimum of two strengths: (1) the strength calculated considering eccentricity of shear on a slab critical section due to combined shear and moment, as prescribed in *ACI 318-95*; and (2) the moment transfer strength equal to $\Sigma M_n/\gamma_f$, where ΣM_n = the sum of positive and negative flexural strengths of a section of slab between lines that are two and one-half slab or drop panel thicknesses ($2.5h$) outside opposite faces of the column or capital; γ_f = the fraction of the moment resisted by flexure per *ACI 318-95*; and h = slab thickness.

For moment about an axis parallel to the slab edge at exterior connections without transverse beams, where the shear on the slab critical section due to gravity loads does not exceed $0.75V_c$, or the shear at a corner support

Table 6-13 Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Two-Way Slabs and Slab-Column Connections

Conditions		Modeling Parameters[4]			Acceptance Criteria[4]				
					Plastic Rotation Angle, radians				
					Component Type				
					Primary			Secondary	
		Plastic Rotation Angle, radians		Residual Strength Ratio	Performance Level				
		a	b	c	IO	LS	CP	LS	CP
i. Slabs controlled by flexure, and slab-column connections[1]									
$\frac{V_g}{V_o}$ [2]	Continuity Reinforcement[3]								
≤ 0.2	Yes	0.02	0.05	0.2	0.01	0.015	0.02	0.03	0.05
≥ 0.4	Yes	0.0	0.04	0.2	0.0	0.0	0.0	0.03	0.04
≤ 0.2	No	0.02	0.02	–	0.01	0.015	0.02	0.015	0.02
≥ 0.4	No	0.0	0.0	–	0.0	0.0	0.0	0.0	0.0
ii. Slabs controlled by inadequate development or splicing along the span[1]									
		0.0	0.02	0.0	0.0	0.0	0.0	0.01	0.02
iii. Slabs controlled by inadequate embedment into slab-column joint[1]									
		0.015	0.03	0.2	0.01	0.01	0.015	0.02	0.03

1. When more than one of the conditions i, ii, and iii occurs for a given component, use the minimum appropriate numerical value from the table.
2. V_g = the gravity shear acting on the slab critical section as defined by ACI 318; V_o = the direct punching shear strength as defined by ACI 318.
3. Under the heading "Continuity Reinforcement," assume "Yes" where at least one of the main bottom bars in each direction is effectively continuous through the column cage. Where the slab is post-tensioned, assume "Yes" where at least one of the post-tensioning tendons in each direction passes through the column cage. Otherwise, assume "No."
4. Interpolation between values shown in the table is permitted.

does not exceed 0.5 V_c, the moment transfer strength may be taken as equal to the flexural strength of a section of slab between lines that are a distance, c_1, outside opposite faces of the column or capital. V_c is the direct punching shear strength defined by *ACI 318-95*.

6.5.4.4 Acceptance Criteria

A. Linear Static and Dynamic Procedures

All component actions shall be classified as being either deformation-controlled or force-controlled, as defined in Chapter 3. In primary components, deformation-controlled actions shall be restricted to flexure in slabs and columns, and shear and moment transfer in slab-column connections. In secondary components, deformation-controlled actions shall also be permitted in shear and reinforcement development, as identified in Table 6-14. All other actions shall be defined as being force-controlled actions.

Design actions on components shall be determined as prescribed in Chapter 3. Where the calculated DCR values exceed unity, the following actions preferably shall be determined using limit analysis principles as prescribed in Chapter 3: (1) moments, shears, torsions, and development and splice actions corresponding to development of component strength in slabs and columns; and (2) axial load in columns, considering

Table 6-14 Numerical Acceptance Criteria for Linear Procedures—Two-Way Slabs and Slab-Column Connections

Conditions		m factors				
		Component Type				
		Primary			Secondary	
		Performance Level				
		IO	LS	CP	LS	CP
i. Slabs controlled by flexure, and slab-column connections[1]						
$\frac{V_g}{V_o}$ [2]	Continuity Reinforcement[3]					
≤ 0.2	Yes	2	2	3	3	4
≥ 0.4	Yes	1	1	1	2	3
≤ 0.2	No	2	2	3	2	3
≥ 0.4	No	1	1	1	1	1
ii. Slabs controlled by inadequate development or splicing along the span[1]						
		–	–	–	3	4
iii. Slabs controlled by inadequate embedment into slab-column joint[1]						
		2	2	3	3	4

1. When more than one of the conditions i, ii, and iii occurs for a given component, use the minimum appropriate numerical value from the table.
2. V_g = the gravity shear acting on the slab critical section as defined by ACI 318; V_o = the direct punching shear strength as defined by ACI 318.
3. Under the heading "Continuity Reinforcement," assume "Yes" where at least one of the main bottom bars in each direction is effectively continuous through the column cage. Where the slab is post-tensioned, assume "Yes" where at least one of the post-tensioning tendons in each direction passes through the column cage. Otherwise, assume "No."

likely plastic action in components above the level in question.

Design actions shall be compared with design strengths to determine which components develop their design strengths. Those components that do not reach their design strengths may be assumed to satisfy the performance criteria for those components. Components that reach their design strengths shall be further evaluated according to this section to determine performance acceptability.

Where the average of the DCRs of columns at a level exceeds the average value of slabs at the same level, and exceeds the greater of 1.0 and $m/2$, the element is defined as a weak story element. In this case, follow the procedure for weak story elements described in Section 6.5.2.4A.

Calculated component actions shall satisfy the requirements of Chapter 3. Tables 6-11 and 6-14 present m values.

B. Nonlinear Static and Dynamic Procedures

Inelastic response shall be restricted to those components and actions listed in Tables 6-7 and 6-13, except where it is demonstrated that other inelastic action can be tolerated considering the selected Performance Levels.

Calculated component actions shall satisfy the requirements of Chapter 3. Maximum permissible inelastic deformations are listed in Tables 6-7 and 6-13. Where inelastic action is indicated for a component or action not listed in these tables, the performance shall be deemed unacceptable. Alternative approaches or values are permitted where justified by experimental evidence and analysis.

6.5.4.5 Rehabilitation Measures

Rehabilitation measures include the general approaches listed in Section 6.5.2.5, plus other approaches based on rational procedures.

Rehabilitated frames shall be evaluated according to the general principles and requirements of this chapter. The effects of rehabilitation on stiffness, strength, and deformability shall be taken into account in an analytical model of the rehabilitated building. Connections required between existing and new elements shall satisfy requirements of Section 6.4.6 and other requirements of the *Guidelines*.

6.6 Precast Concrete Frames

6.6.1 Types of Precast Concrete Frames

Precast concrete frames are those elements that are constructed from individually made beams and columns, that are assembled to create gravity-load-carrying systems. These systems are sometimes expected to directly resist lateral loads, and are always required to deform in a manner that is compatible with the structure as a whole.

The provisions of this section are applicable to precast concrete frames that emulate cast-in-place moment frames, precast concrete beam-column moment frames other than emulated cast-in-place moment frames, and precast concrete frames not expected to directly resist lateral loads.

6.6.1.1 Precast Concrete Frames that Emulate Cast-in-Place Moment Frames

Emulated moment frames of precast concrete are those precast beam-column systems that are interconnected using reinforcing and wet concrete in such a way as to create a system that will act to resist lateral loads in a manner similar to cast-in-place concrete systems. These systems are recognized and accepted by the 1994 *NEHRP Recommended Provisions* (BSSC, 1995), and are based on ACI 318, which requires safety and serviceability levels expected from monolithic construction. There are insufficient research and testing data at this time to qualify systems assembled using dry joints as emulated moment frames.

6.6.1.2 Precast Concrete Beam-Column Moment Frames other than Emulated Cast-in-Place Moment Frames

Frames of this classification are assembled using dry joints; that is, connections are made by bolting, welding, post-tensioning, or other similar means. Frames of this nature may act alone to resist lateral loads, or they may act in conjunction with shear walls, braced frames, or other elements to form a dual system. The appendix to Chapter 6 of the 1994 *NEHRP Recommended Provisions* (BSSC, 1995) contains a trial version of code provisions for new construction of this nature, but it was felt to be premature in 1994 to base actual provisions on the material in the appendix.

6.6.1.3 Precast Concrete Frames Not Expected to Resist Lateral Loads Directly

Frames of this classification are assembled using dry joints similar to those of Section 6.6.1.2, but are not expected to participate in resisting the lateral loads directly or significantly. Shear walls, braced frames, or steel moment frames are expected to provide the entire lateral load resistance, but the precast concrete “gravity” frame system must be able to deform in a manner that is compatible with the structure as a whole. Conservative assumptions shall be made concerning the relative fixity of joints.

6.6.2 Precast Concrete Frames that Emulate Cast-in-Place Moment Frames

6.6.2.1 General Considerations

The analysis model for an emulated beam-column frame element shall represent strength, stiffness, and deformation capacity of beams, columns, beam-column joints, and other components that may be part of the frame. Potential failure in flexure, shear, and reinforcement development at any section along the component length shall be considered. Interaction with other elements, including nonstructural elements and components, shall be included. All other considerations of Section 6.5.2.1 shall be taken into account. In addition, special care shall be taken to consider the effects of shortening due to creep, and prestressing and post-tensioning on member behavior.

6.6.2.2 Stiffness for Analysis

Stiffness for analysis shall be as defined in Section 6.5.2.2. The effects of prestressing shall be considered when computing the effective stiffness values using Table 6-4.

6.6.2.3 Design Strengths

Component strength shall be computed according to the requirements of Section 6.5.2.3, with the additional requirement that the following factors be included in the calculation of strength:

1. Effects of prestressing that are present, including, but not limited to, reduction in rotation capacity, secondary stresses induced, and amount of effective prestress force remaining
2. Effects of construction sequence, including the possibility that the moment connections may have been constructed after dead load had been applied to portions of the structure
3. Effects of restraint that may be present due to interaction with interconnected wall or brace components

6.6.2.4 Acceptance Criteria

Acceptance criteria for precast concrete frames that emulate cast-in-place moment frames are as described in Section 6.5.2.4, except that the factors defined in Section 6.6.2.3 shall also be considered.

6.6.2.5 Rehabilitation Measures

Rehabilitation measures for emulated cast-in-place moment frames are given in Section 6.5.2.5. Special consideration shall be given to the presence of prestressing strand when installing new elements and when adding new rigid elements to the existing system.

6.6.3 Precast Concrete Beam-Column Moment Frames other than Emulated Cast-in-Place Moment Frames

6.6.3.1 General Considerations

The analysis model for precast concrete beam-column moment frames other than emulated moment frames shall be established following Section 6.5.2.1 for reinforced concrete beam-column moment frames, with additional consideration of the special nature of the dry joints used in assembling the precast system. The requirements given in the appendix to Chapter 6 of the 1994 *NEHRP Recommended Provisions* for this type of structural system should be adhered to where possible, and the philosophy and approach should be employed when designing new connections for existing components. See also Section 6.4.6.

6.6.3.2 Stiffness for Analysis

Stiffness for analysis shall be as defined in Sections 6.5.2.2 and 6.6.2.2. Flexibilities associated with connections should be included in the analytical model. See also Section 6.4.6.

6.6.3.3 Design Strengths

Component strength shall be computed according to the requirements of Sections 6.5.2.3 and 6.6.2.3, with the additional requirements that the connections comply with the appendix to Chapter 6 of the 1994 *NEHRP Recommended Provisions*, and connection strength shall be represented. See also Section 6.4.6.

6.6.3.4 Acceptance Criteria

Acceptance criteria for precast concrete beam-column moment frames other than emulated cast-in-place moment frames are given in Sections 6.5.2.4 and 6.6.2.4, with the additional requirement that the connections meet the requirements of Section 6.A.4 of the appendix to Chapter 6 of the 1994 *NEHRP Recommended Provisions*. See also Section 6.4.6.

6.6.3.5 Rehabilitation Measures

Rehabilitation measures for the frames of this section shall meet the requirements of Section 6.6.2.5. Special consideration shall be given to connections that are stressed beyond their elastic limit.

6.6.4 Precast Concrete Frames Not Expected to Resist Lateral Loads Directly

6.6.4.1 General Considerations

The analysis model for precast concrete frames that are not expected to resist significant lateral loads directly shall include the effects of deformations that the lateral-load-resisting system will experience. The general considerations of Sections 6.5.2.1 and 6.6.3.1 shall be included.

6.6.4.2 Stiffness for Analysis

The stiffness for analysis considers possible resistance that may develop under lateral deformation. In some cases it may be appropriate to assume zero lateral stiffness. However, the Northridge earthquake graphically demonstrated that there are practically no situations where the precast column can be considered to be completely pinned top and bottom, and as a consequence, not resisting any shear from building drift. Several parking structures collapsed as a result of this defect. Conservative assumptions should be made.

6.6.4.3 Design Strengths

Component strength shall be computed according to the requirements of Section 6.6.3.3. All components shall have sufficient strength and ductility to transmit induced forces from one member to another and to the designated lateral-force-resisting system.

6.6.4.4 Acceptance Criteria

Acceptance criteria for components in precast concrete frames not expected to directly resist lateral loads are given in Section 6.6.3.4. All moments, shear forces, and axial loads induced through the deformation of the intended lateral-force-resisting system shall be checked for acceptability by appropriate criteria in the referenced section.

6.6.4.5 Rehabilitation Measures

Rehabilitation measures for the frames discussed in this section shall meet the requirements of Section 6.6.3.5.

6.7 Concrete Frames with Infills

6.7.1 Types of Concrete Frames with Infills

Concrete frames with infills are those frames constructed with complete gravity-load-carrying frames infilled with masonry or concrete, constructed in such a way that the infill and the concrete frame interact when subjected to design load combinations.

Infills may be considered to be isolated infills if they are isolated from the surrounding frame according to the minimum gap requirements described in Section 7.5.1. If all infills in a frame are isolated infills, the frame should be analyzed as an isolated frame according to provisions given elsewhere in this chapter, and the isolated infill panels shall be analyzed according to the requirements of Chapter 7.

The provisions are applicable to frames with existing infills, frames that are rehabilitated by addition or removal of material, and concrete frames that are rehabilitated by the addition of new infills.

6.7.1.1 Types of Frames

The provisions are applicable to frames that are cast monolithically and frames that are precast. Types of concrete frames are described in Sections 6.5, 6.6, and 6.10.

6.7.1.2 Masonry Infills

Types of masonry infills are described in Chapter 7.

6.7.1.3 Concrete Infills

The construction of concrete-infilled frames is very similar to that for masonry-infilled frames, except that the infill is of concrete instead of masonry units. In older existing buildings, the concrete infill commonly contains nominal reinforcement, which is unlikely to extend into the surrounding frame. The concrete is likely to be of lower quality than that used in the frame, and should be investigated separately from investigations of the frame concrete.

6.7.2 Concrete Frames with Masonry Infills

6.7.2.1 General Considerations

The analysis model for a concrete frame with masonry infills shall be sufficiently detailed to represent strength, stiffness, and deformation capacity of beams, slabs, columns, beam-column joints, masonry infills, and all connections and components that may be part of the element. Potential failure in flexure, shear, anchorage, reinforcement development, or crushing at any section shall be considered. Interaction with other nonstructural elements and components shall be included.

Behavior of a concrete frame with masonry infill resisting lateral forces within its plane may be calculated based on linear elastic behavior if it can be demonstrated that the wall will not crack when subjected to design lateral forces. In this case, the assemblage of frame and infill should be considered to be a homogeneous medium for stiffness computations.

Behavior of cracked concrete frames with masonry infills may be represented by a diagonally braced frame model in which the columns act as vertical chords, the beams act as horizontal ties, and the infill is modeled using the equivalent compression strut analogy. Requirements for the equivalent compression strut analogy are described in Chapter 7.

Frame components shall be evaluated for forces imparted to them through interaction of the frame with the infill, as specified in Chapter 7. In frames with full-height masonry infills, the evaluation shall include the effect of strut compression forces applied to the column and beam, eccentric from the beam-column joint. In frames with partial-height masonry infills, the evaluation shall include the reduced effective length of the columns in the noninfilled portion of the bay.

In frames having infills in some bays and no infill in other bays, the restraint of the infill shall be represented as described above, and the noninfilled bays shall be modeled as frames according to the specifications of this chapter. Where infills create a discontinuous wall, the effects on overall building performance shall be considered.

6.7.2.2 Stiffness for Analysis

A. Linear Static and Dynamic Procedures

General aspects of modeling are described in Section 6.7.2.1. Beams and columns in infilled portions may be modeled considering axial tension and compression flexibilities only. Noninfilled portions shall be modeled according to procedures described for noninfilled frames. Effective stiffnesses shall be according to Section 6.4.1.2.

B. Nonlinear Static Procedure

Nonlinear load-deformation relations shall follow the general guidelines of Section 6.4.1.2.

Beams and columns in infilled portions may be modeled using nonlinear truss elements. Beams and columns in noninfilled portions may be modeled using procedures described in this chapter. The model shall be capable of representing inelastic response along the component lengths.

Monotonic load-deformation relations shall be according to the generalized relation shown in Figure 6-1, except different relations are permitted where verified by tests. Numerical quantities in Figure 6-1 may be derived from tests or rational analyses following the general guidelines of Chapter 2, and shall take into account the interactions between frame and infill components. Alternatively, the following may be used for monolithic reinforced concrete frames.

1. For beams and columns in noninfilled portions of frames, where the generalized deformation is taken as rotation in the flexural plastic hinge zone, the plastic hinge rotation capacities shall be as defined by Table 6-17.

2. For masonry infills, the generalized deformations and control points shall be as defined in Chapter 7.

3. For beams and columns in infilled portions of frames, where the generalized deformation is taken as elongation or compression displacement of the beams or columns, the tension and compression strain capacities shall be as specified in Table 6-15.

C. Nonlinear Dynamic Procedure

For the NDP, the complete hysteresis behavior of each component shall be modeled using properties verified by tests. Unloading and reloading properties shall represent significant stiffness and strength degradation characteristics.

6.7.2.3 Design Strengths

Strengths of reinforced concrete components shall be according to the general requirements of Section 6.4.2, as modified by other specifications of this chapter. Strengths of masonry infills shall be according to the requirements of Chapter 7. Strengths shall consider limitations imposed by beams, columns, and joints in unfilled portions of frames; tensile and compressive capacity of columns acting as boundary elements of infilled frames; local forces applied from the infill to the frame; strength of the infill; and connections with adjacent elements. .

Table 6-15 Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Reinforced Concrete Infilled Frames

	Modeling Parameters[4]			Acceptance Criteria				
				Total Strain				
				Component Type				
			Residual Strength Ratio	Primary			Secondary	
	Total Strain			Performance Level				
Conditions	**d**	**e**	**c**	**IO**	**LS**	**CP**	**LS**	**CP**
i. Columns modeled as compression chords[3]								
Columns confined along entire length[2]	0.02	0.04	0.4	0.003	0.015	0.020	0.03	0.04
All other cases	0.003	0.01	0.2	0.002	0.002	0.003	0.01	0.01
ii. Columns modeled as tension chords[3]								
Columns with well-confined splices, or no splices	0.05	0.05	0.0	0.01	0.03	0.04	0.04	0.05
All other cases	See note 1	0.03	0.2	See note 1			0.02	0.03

1. Splice failure in a primary component can result in loss of lateral load resistance. For these cases, refer to the generalized procedure of Section 6.4.2. For primary actions, Collapse Prevention Performance Level shall be defined as the deformation at which strength degradation begins. Life Safety Performance Level shall be taken as three-quarters of that value.

2 A column may be considered to be confined along its entire length when the quantity of transverse reinforcement along the entire story height including the joint is equal to three-quarters of that required by ACI 318 for boundary elements of concrete shear walls. The maximum longitudinal spacing of sets of hoops shall not exceed $h/3$ nor $8d_b$.

3. In most infilled walls, load reversals will result in both conditions i and ii applying to a single column, but for different loading directions.

4. Interpolation is not permitted.

6.7.2.4 Acceptance Criteria

A. Linear Static and Dynamic Procedures

All component actions shall be classified as either deformation-controlled or force-controlled, as defined in Chapter 3. In primary components, deformation-controlled actions shall be restricted to flexure and axial actions in beams, slabs, and columns, and lateral deformations in masonry infill panels. In secondary components, deformation-controlled actions shall be restricted to those actions identified for the isolated frame in this chapter and for the masonry infill in Chapter 7.

Design actions shall be determined as prescribed in Chapter 3. Where calculated DCR values exceed unity, the following actions preferably shall be determined using limit analysis principles as prescribed in Chapter 3: (1) moments, shears, torsions, and development and splice actions corresponding to development of component strength in beams, columns, or masonry infills; and (2) column axial load corresponding to development of the flexural capacity of the infilled frame acting as a cantilever wall.

Design actions shall be compared with design strengths to determine which components develop their design strengths. Those components that have design actions less than design strengths may be assumed to satisfy the performance criteria for those components. Components that reach their design strengths shall be further evaluated according to this section to determine performance acceptability.

Calculated component actions shall satisfy the requirements of Chapter 3. Refer to Section 7.5.2.2 for *m* values for masonry infills. Refer to other sections of this chapter for *m* values for concrete frames; *m* values for columns modeled as tension and compression chords are in Table 6-16.

Table 6-16 Numerical Acceptance Criteria for Linear Procedures—Reinforced Concrete Infilled Frames

	m factors[3]				
	Component Type				
	Primary			Secondary	
	Performance Level				
Conditions	**IO**	**LS**	**CP**	**LS**	**CP**
i. Columns modeled as compression chords[2]					
Columns confined along entire length[1]	1	3	4	4	5
All other cases	1	1	1	1	1
ii. Columns modeled as tension chords[2]					
Columns with well-confined splices, or no splices	3	4	5	5	6
All other cases	1	2	2	3	4

1. A column may be considered to be confined along its entire length when the quantity of transverse reinforcement along the entire story height including the joint is equal to three-quarters of that required by ACI 318 for boundary elements of concrete shear walls. The maximum longitudinal spacing of sets of hoops shall not exceed $h/3$ nor $8d_b$.
2. In most infilled walls, load reversals will result in both conditions i and ii applying to a single column, but for different loading directions.
3. Interpolation is not permitted.

B. Nonlinear Static and Dynamic Procedures

Inelastic response shall be restricted to those components and actions that are permitted for isolated frames in this chapter and for masonry infills in Chapter 7.

Design actions shall be compared with design strengths to determine which components develop their design strengths. Those components that have design actions less than design strengths may be assumed to satisfy the performance criteria for those components. Components that reach their design strengths shall be further evaluated, according to Section 6.5.2.4B, to determine performance acceptability.

Calculated component actions shall not exceed the numerical values listed in Table 6-15, the relevant tables for isolated frames given in this chapter, and the relevant tables for masonry infills given in Chapter 7. Where inelastic action is indicated for a component or action not listed in Tables 6-10 through 6-12, the performance shall be deemed unacceptable. Alternative approaches or values are permitted where justified by experimental evidence and analysis.

6.7.2.5 Rehabilitation Measures

Rehabilitation measures include the general approaches listed for isolated frames in this chapter, the measures listed for masonry infills in Section 7.5, and other approaches based on rational procedures. Both in-plane and out-of-plane loading shall be considered.

The following methods should be considered.

- **Jacketing existing beams, columns, or joints with new reinforced concrete, steel, or fiber wrap overlays.** The new materials shall be designed and constructed to act compositely with the existing concrete. Where reinforced concrete jackets are used, the design shall provide detailing to enhance ductility. Component strength shall be taken to not exceed any limiting strength of connections with adjacent components. Jackets designed to provide increased connection strength and improved continuity between adjacent components are permitted.

- **Post-tensioning existing beams, columns, or joints using external post-tensioned reinforcement.** Vertical post-tensioning may be useful for increasing tensile capacity of columns acting as boundary zones. Anchorages shall be located away from regions where inelastic action is anticipated, and shall be designed considering possible force variations due to earthquake loading.

- **Modification of the element by selective material removal from the existing element.** Either the infill can be completely removed from the frame, or gaps can be provided between the frame and the infill. In the latter case, the gap requirements of Chapter 7 shall be satisfied.

- **Improvement of deficient existing reinforcement details.** This approach involves removal of cover concrete, modification of existing reinforcement details, and casting of new cover concrete. Concrete removal shall avoid unintended damage to core concrete and the bond between existing reinforcement and core concrete. New cover concrete shall be designed and constructed to achieve fully composite action with the existing materials.

- **Changing the building system to reduce the demands on the existing element.** Examples include the addition of supplementary lateral-force-resisting elements such as walls, steel braces, or buttresses; seismic isolation; and mass reduction.

Rehabilitated frames shall be evaluated according to the general principles and requirements of this chapter. The effects of rehabilitation on stiffness, strength, and deformability shall be taken into account in the analytical model. Connections required between existing and new elements shall satisfy the requirements of Section 6.4.6 and other requirements of the *Guidelines*.

6.7.3 Concrete Frames with Concrete Infills

6.7.3.1 General Considerations

The analysis model for a concrete frame with concrete infills shall be sufficiently detailed to represent the strength, stiffness, and deformation capacity of beams, slabs, columns, beam-column joints, concrete infills, and all connections and components that may be part of the elements. Potential failure in flexure, shear, anchorage, reinforcement development, or crushing at any section shall be considered. Interaction with other nonstructural elements and components shall be included.

The numerical model should be established considering the relative stiffness and strength of the frame and the infill, as well as the level of deformations and associated damage. For low deformation levels, and for cases where the frame is relatively flexible, it may be suitable to model the infilled frame as a solid shear wall, although openings should be considered where they occur. In other cases, it may be more suitable to model the frame-infill system using a braced-frame analogy such as that described for concrete frames with masonry infills in Section 6.7.2. Some judgment is necessary to determine the appropriate type and complexity of the analytical model.

Frame components shall be evaluated for forces imparted to them through interaction of the frame with the infill, as specified in Chapter 7. In frames with full-height infills, the evaluation shall include the effect of strut compression forces applied to the column and beam eccentric from the beam-column joint. In frames with partial-height infills, the evaluation shall include the reduced effective length of the columns in the noninfilled portion of the bay.

In frames having infills in some bays and no infills in other bays, the restraint of the infill shall be represented as described above, and the noninfilled bays shall be modeled as frames according to the specifications of this chapter. Where infills create a discontinuous wall, the effects on overall building performance shall be considered.

6.7.3.2 Stiffness for Analysis

A. Linear Static and Dynamic Procedures

General aspects of modeling are described in Section 6.7.3.1. Effective stiffnesses shall be according to the general principles of Section 6.4.1.2.

B. Nonlinear Static Procedure

General aspects of modeling are described in Section 6.7.3.1. Nonlinear load-deformation relations shall follow the general guidelines of Section 6.4.1.2.

Monotonic load-deformation relations shall be according to the generalized relation shown in Figure 6-1, except different relations are permitted where verified by tests. Numerical quantities in Figure 6-1 may be derived from tests or rational analyses following the general guidelines of Section 2.13, and shall take into account the interactions between frame and infill components. The guidelines of Section 6.7.2.2 may be used to guide development of modeling parameters for concrete frames with concrete infills.

C. Nonlinear Dynamic Procedure

For the NDP, the complete hysteresis behavior of each component shall be modeled using properties verified by tests. Unloading and reloading properties shall represent significant stiffness and strength degradation characteristics.

6.7.3.3 Design Strengths

Strengths of reinforced concrete components shall be according to the general requirements of Section 6.4.2, as modified by other specifications of this chapter. Strengths shall consider limitations imposed by beams, columns, and joints in unfilled portions of frames; tensile and compressive capacity of columns acting as boundary elements of infilled frames; local forces applied from the infill to the frame; strength of the infill; and connections with adjacent elements.

Strengths of existing concrete infills shall be determined considering shear strength of the infill panel. For this calculation, procedures specified in Section 6.8.2.3 shall be used for calculation of the shear strength of a wall segment.

Where the frame and concrete infill are assumed to act as a monolithic wall, flexural strength shall be based on continuity of vertical reinforcement in both (1) the columns acting as boundary elements, and (2) the infill wall, including anchorage of the infill reinforcement in the boundary frame.

6.7.3.4 Acceptance Criteria

The acceptance criteria for concrete frames with concrete infills should be guided by relevant acceptance criteria of Sections 6.7.2.4, 6.8, and 6.9.

6.7.3.5 Rehabilitation Measures

Rehabilitation measures include the general approaches listed for masonry infilled frames in Section 6.7.2.5.

Strengthening of the existing infill may be considered as an option for rehabilitation. Shotcrete can be applied to the face of an existing wall to increase the thickness and shear strength. For this purpose, the face of the existing wall should be roughened, a mat of reinforcing steel should be doweled into the existing structure, and shotcrete should be applied to the desired thickness.

Rehabilitated frames shall be evaluated according to the general principles and requirements of this chapter. The effects of rehabilitation on stiffness, strength, and deformability shall be taken into account in the analytical model. Connections required between existing and new elements shall satisfy the requirements of Section 6.4.6 and other requirements of the *Guidelines*.

6.8 Concrete Shear Walls

6.8.1 Types of Concrete Shear Walls and Associated Components

Concrete shear walls consist of planar vertical elements that normally serve as the primary lateral-load-resisting elements when they are used in concrete structures. In general, shear walls (or wall segments) are considered to be slender if their aspect ratio (height/length) is Š 3.0, and they are considered to be short if their aspect ratio is ≤ 1.5. Slender shear walls are normally controlled by flexural behavior; short walls are normally controlled by shear behavior. The response of walls with intermediate aspect ratios is influenced by both flexure and shear.

The provisions given here are applicable to all shear walls in all types of structural systems that incorporate shear walls. This includes isolated shear walls, shear walls used in dual (wall-frame) systems, coupled shear walls, and discontinuous shear walls. Shear walls are considered to be solid walls if they have small openings that do not significantly influence the strength or inelastic behavior of the wall. Perforated shear walls are characterized by a regular pattern of large openings in both horizontal and vertical directions that create a series of pier and deep beam elements. In the discussions and tables that appear in the following sections, these vertical piers and horizontal beams will both be referred to as wall segments.

Provisions are also included for coupling beams and columns that support discontinuous shear walls. These are special frame components that are associated more with shear walls than with the normal frame elements covered in Section 6.5.

6.8.1.1 Monolithic Reinforced Concrete Shear Walls and Wall Segments

Monolithic reinforced concrete (RC) shear walls consist of vertical cast-in-place elements, usually with a constant cross section, that typically form open or closed shapes around vertical building shafts. Shear walls are also used frequently along portions of the perimeter of the building. The wall reinforcement is normally continuous in both the horizontal and vertical directions, and bars are typically lap spliced for tension continuity. The reinforcement mesh may also contain horizontal ties around vertical bars that are concentrated either near the vertical edges of a wall with constant thickness, or in boundary members formed at the wall edges. The amount and spacing of these ties is important for determining how well the concrete at the wall edge is confined, and thus for determining the lateral deformation capacity of the wall.

In general, slender reinforced concrete shear walls will be governed by flexure and will tend to form a plastic flexural hinge near the base of the wall under severe lateral loading. The ductility of the wall will be a function of the percentage of longitudinal reinforcement concentrated near the boundaries of the wall, the level of axial load, the amount of lateral shear required to cause flexural yielding, and the thickness and reinforcement used in the web portion of the shear wall. In general, higher axial load stresses and higher shear stresses will reduce the flexural ductility and energy absorbing capability of the shear wall. Squat shear walls will normally be governed by shear. These walls will normally have a limited ability to deform beyond the elastic range and continue to carry lateral loads. Thus, these walls are typically designed either as displacement-controlled components with low ductility capacities or as force-controlled components.

Shear walls or wall segments with axial loads greater than 0.35 P_o shall not be considered effective in resisting seismic forces. The maximum spacing of horizontal and vertical reinforcement shall not exceed 18 inches. Walls with horizontal and vertical reinforcement ratios less than 0.0025, but with reinforcement spacings less than 18 inches, shall be permitted where the shear force demand does not exceed the reduced nominal shear strength of the wall calculated in accordance with Section 6.8.2.3.

6.8.1.2 Reinforced Concrete Columns Supporting Discontinuous Shear Walls

In shear wall buildings it is not uncommon to find that some walls are terminated either to create commercial space in the first story or to create parking spaces in the basement. In such cases, the walls are commonly supported by columns. Such designs are not recommended in seismic zones because very large demands may be placed on these columns during earthquake loading. In older buildings such columns will often have “standard” longitudinal and transverse reinforcement; the behavior of such columns during past earthquakes indicates that tightly spaced closed ties with well-anchored 135-degree hooks will be required for the building to survive severe earthquake loading.

6.8.1.3 Reinforced Concrete Coupling Beams

Reinforced concrete coupling beams are used to link two shear walls together. The coupled walls are generally much stiffer and stronger than they would be if they acted independently. Coupling beams typically have a small span-to-depth ratio, and their inelastic behavior is normally affected by the high shear forces acting in these components. Coupling beams in most older reinforced concrete buildings will commonly have “conventional” reinforcement that consists of longitudinal flexural steel and transverse steel for shear. In some, more modern buildings, or in buildings where coupled shear walls are used for seismic rehabilitation, the coupling beams may use diagonal reinforcement as the primary reinforcement for both flexure and shear. The inelastic behavior of coupling beams that use diagonal reinforcement has been shown experimentally to be much better with respect to retention of strength, stiffness, and energy dissipation capacity than the observed behavior of coupling beams with conventional reinforcement.

6.8.2 Reinforced Concrete Shear Walls, Wall Segments, Coupling Beams, and RC Columns Supporting Discontinuous Shear Walls

6.8.2.1 General Modeling Considerations

The analysis model for an RC shear wall element shall be sufficiently detailed to represent the stiffness, strength, and deformation capacity of the overall shear wall. Potential failure in flexure, shear, and reinforcement development at any point in the shear wall shall be considered. Interaction with other structural and nonstructural elements shall be included.

In most cases, shear walls and wall elements may be modeled analytically as equivalent beam-column elements that include both flexural and shear deformations. The flexural strength of beam-column elements shall include the interaction of axial load and bending. The rigid connection zone at beam connections to this equivalent beam-column element will need to be long enough to properly represent the distance from the wall centroid—where the beam-column element is placed in the computer model—to the edge of the wall. Unsymmetrical wall sections shall model the different bending capacities for the two loading directions.

For rectangular shear walls and wall segments with $h_w / l_w \leq 2.5$, and flanged wall sections with $h_w / l_w \leq 3.5$, shear deformations become more significant. For such cases, either a modified beam-column analogy or a multiple-node, multiple-spring approach should be used (references are given in the *Commentary*). Because shear walls usually respond in single curvature over a story height, the use of one multiple-spring element per story is recommended for modeling shear walls. For wall segments, which typically deform into a double curvature pattern, the beam-column element is usually preferred. If a multiple-spring model is used for a wall segment, then it is recommended that two elements be used over the length of the wall segment.

A beam element that incorporates both bending and shear deformations shall be used to model coupling beams. It is recommended that the element inelastic response should account for the loss of shear strength and stiffness during reversed cyclic loading to large deformations. Coupling beams that have diagonal reinforcement satisfying the 1994 *NEHRP Recommended Provisions* (BSSC, 1995) will commonly have a stable hysteretic response under large load reversals. Therefore, these members could adequately be modeled with beam elements used for typical frame analyses.

Columns supporting discontinuous shear walls may be modeled with beam-column elements typically used in frame analysis. This element should also account for shear deformations, and care must be taken to ensure that the model properly reflects the potentially rapid reduction in shear stiffness and strength these columns may experience after the onset of flexural yielding.

The diaphragm action of concrete slabs that interconnect shear walls and frame columns shall be properly represented.

6.8.2.2 Stiffness for Analysis

The stiffness of all the elements discussed in this section depends on the material properties, component dimensions, reinforcement quantities, boundary conditions, and current state of the member with respect to cracking and stress levels. All of these aspects should be considered when defining the effective stiffness of an element. General values for effective stiffness are given in Table 6-4. To obtain a proper distribution of lateral forces in bearing wall buildings, all of the walls shall be assumed to be either cracked or uncracked. In buildings where lateral load resistance is provided by either structural walls only, or a combination of walls and frame members, all shear walls and wall segments discussed in this section should be considered to be cracked.

For coupling beams, the values given in Table 6-4 for nonprestressed beams should be used. Columns supporting discontinuous shear walls will experience significant changes in axial load during lateral loading of the shear wall they support. Thus, the stiffness values for these column elements will need to change between the values given for columns in tension and compression, depending on the direction of the lateral load being resisted by the shear wall.

A. Linear Static and Dynamic Procedures

Shear walls and associated components shall be modeled considering axial, flexural, and shear stiffness. For closed and open wall shapes, such as box, T, L, I, and C sections, the effective tension or compression flange widths on each side of the web shall be taken as the smaller of: (1) one-fifth of the wall height, (2) half the distance to the next web, or (3) the provided width of the flange. The calculated stiffnesses to be used in analysis shall be in accordance with the requirements of Section 6.4.1.2.

Joints between shear walls and frame elements shall be modeled as stiff components and shall be considered rigid in most cases.

B. Nonlinear Static and Dynamic Procedures

Nonlinear load-deformation relations shall follow the general procedures described in Section 6.4.1.2.

Monotonic load-deformation relationships for analytical models that represent shear walls, wall elements, coupling beams, and RC columns that support discontinuous shear walls shall be of the general shapes defined in Figure 6-1. For both of the load-deformation relationships in Figure 6-1, point *B* corresponds to significant yielding, point *C* corresponds to the point where significant lateral resistance is assumed to be lost, and point *E* corresponds to the point where gravity load resistance is assumed to be lost.

The load-deformation relationship in Figure 6-1(a) should be referred to for shear walls and wall segments having inelastic behavior under lateral loading that is governed by flexure, as well as columns supporting discontinuous shear walls. For all of these members, the x-axis of Figure 6-1(a) should be taken as the rotation over the plastic hinging region at the end of the member (Figure 6-2). The hinge rotation at point *B* corresponds to the yield point, θ_y, and is given by the following expression:

$$\theta_y = \left(\frac{M_y}{E_c I}\right) l_p \tag{6-5}$$

where:

M_y = Yield moment capacity of the shear wall or wall segment

E_c = Concrete modulus

I = Member moment of inertia, as discussed above

l_p = Assumed plastic hinge length

For analytical models of shear walls and wall segments, the value of l_p shall be set equal to 0.5 times the flexural depth of the element, but less than one story height for shear walls and less than 50% of the element length for wall segments. For RC columns supporting discontinuous shear walls, l_p shall be set equal to 0.5 times the flexural depth of the component.

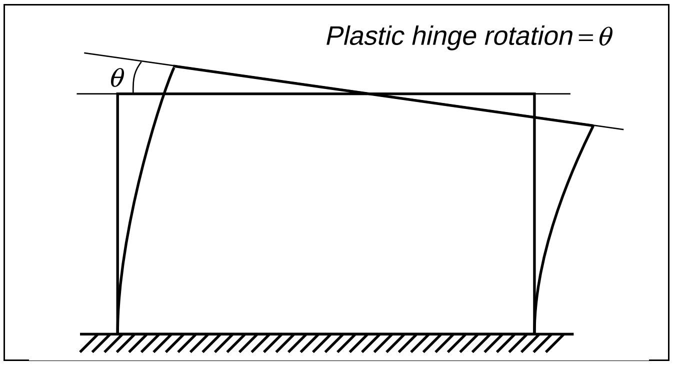

Figure 6-2 ***Plastic Hinge Rotation in Shear Wall where Flexure Dominates Inelastic Response***

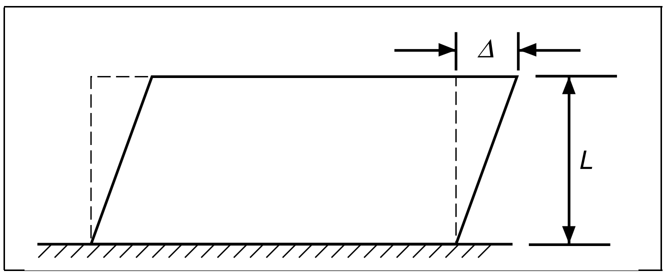

Figure 6-3 ***Story Drift in Shear Wall where Shear Dominates Inelastic Response***

Values for the variables *a*, *b*, and *c*, which are required to define the location of points *C*, *D*, and *E* in Figure 6-1(a), are given in Table 6-17.

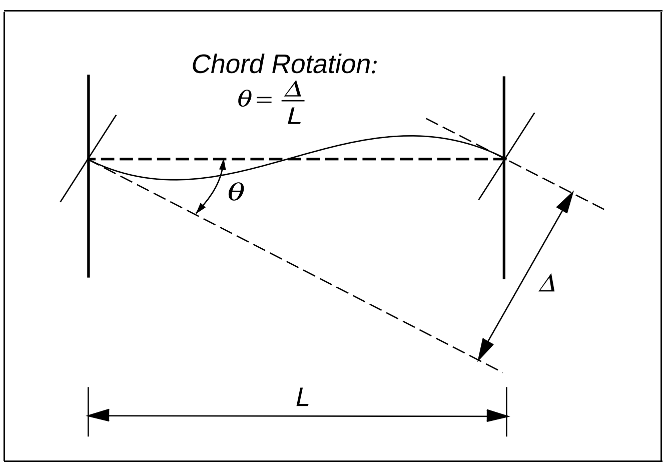

Figure 6-4 ***Chord Rotation for Shear Wall Coupling Beams***

For shear walls and wall segments whose inelastic response is controlled by shear, it is more appropriate to use drift as the deformation value in Figure 6-1(b). For shear walls, this drift is actually the story drift as shown in Figure 6-3. For wall segments, Figure 6-3 essentially represents the member drift.

For coupling beams, the deformation measure to be used in Figure 6-1(b) is the chord rotation for the member, as defined in Figure 6-4. Chord rotation is the most representative measure of the deformed state of a coupling beam, whether its inelastic response is governed by flexure or by shear.

Values for the variables *d*, *e*, and *c*, which are required to find the points *C*, *D*, and *E* in Figure 6-1(b), are given in Tables 6-17 and 6-18 for the appropriate members. Linear interpolation between tabulated values shall be used if the member under analysis has conditions that are between the limits given in the tables.

For the NDP, the complete hysteresis behavior of each component shall be modeled using properties verified by experimental evidence. The relationships in Figure 6-1 may be taken to represent the envelope for the analysis. The unloading and reloading stiffnesses and strengths, and any pinching of the load-versus-rotation hysteresis loops, shall reflect the behavior experimentally observed for wall elements similar to the one under investigation.

6.8.2.3 Design Strengths

The discussions in the following paragraphs shall apply to shear walls, wall segments, coupling beams, and columns supporting discontinuous shear walls. In general, component strengths shall be computed according to the general requirements of Section 6.4.2, except as modified here. The yield and maximum component strength shall be determined considering the potential for failure in flexure, shear, or development under combined gravity and lateral load.

Nominal flexural strength of shear walls or wall segments shall be determined using the fundamental principles given in Chapter 10 of *Building Code Requirements for Structural Concrete, ACI 318-95* (ACI, 1995). For calculation of nominal flexural strength, the effective compression and tension flange widths defined in Section 6.8.2.2A shall be used, except that the first limit shall be changed to one-tenth of the wall height. When determining the flexural yield strength of a shear wall, as represented by point *B* in

Table 6-17 Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Members Controlled by Flexure

			Plastic Hinge Rotation (radians)		**Residual Strength Ratio**	**Acceptable Plastic Hinge Rotation (radians)**				
						Component Type				
						Primary			**Secondary**	
						Performance Level				
Conditions			**a**	**b**	**c**	**IO**	**LS**	**CP**	**LS**	**CP**
i. Shear walls and wall segments										
$\frac{(A_s - A'_s)f_y + P}{t_w l_w f'_c}$	$\frac{\text{Shear}}{t_w l_w \sqrt{f'_c}}$	Confined Boundary[1]								
≤ 0.1	≤3	Yes	0.015	0.020	0.75	0.005	0.010	0.015	0.015	0.020
≤ 0.1	≥ 6	Yes	0.010	0.015	0.40	0.004	0.008	0.010	0.010	0.015
≥ 0.25	≤ 3	Yes	0.009	0.012	0.60	0.003	0.006	0.009	0.009	0.012
≥ 0.25	≥ 6	Yes	0.005	0.010	0.30	0.001	0.003	0.005	0.005	0.010
≤ 0.1	≤ 3	No	0.008	0.015	0.60	0.002	0.004	0.008	0.008	0.015
≤ 0.1	≥ 6	No	0.006	0.010	0.30	0.002	0.004	0.006	0.006	0.010
≥ 0.25	≤ 3	No	0.003	0.005	0.25	0.001	0.002	0.003	0.003	0.005
≥ 0.25	≥ 6	No	0.002	0.004	0.20	0.001	0.001	0.002	0.002	0.004
ii. Columns supporting discontinuous shear walls										
Transverse reinforcement[2]										
Conforming			0.010	0.015	0.20	0.003	0.007	0.010	n.a.	n.a.
Nonconforming			0.0	0.0	0.0	0.0	0.0	0.0	n.a.	n.a.
			Chord Rotation (radians)							
			d	**e**						
iii. Shear wall coupling beams										
Longitudinal reinforcement and transverse reinforcement[3]		$\frac{\text{Shear}}{t_w l_w \sqrt{f'_c}}$								
Conventional longitudinal reinforcement with conforming transverse reinforcement		≤ 3	0.025	0.040	0.75	0.006	0.015	0.025	0.025	0.040
		≥ 6	0.015	0.030	0.50	0.005	0.010	0.015	0.015	0.030
Conventional longitudinal reinforcement with nonconforming transverse reinforcement		≤ 3	0.020	0.035	0.50	0.006	0.012	0.020	0.020	0.035
		≥ 6	0.010	0.025	0.25	0.005	0.008	0.010	0.010	0.025
Diagonal reinforcement		n.a.	0.030	0.050	0.80	0.006	0.018	0.030	0.030	0.050

1. Requirements for a confined boundary are the same as those given in *ACI 318-95*.
2. Requirements for conforming transverse reinforcement are: (a) closed stirrups over the entire length of the column at a spacing ≤ $d/2$, and (b) strength of closed stirrups $V_s \geq$ required shear strength of column.
3. Conventional longitudinal reinforcement consists of top and bottom steel parallel to the longitudinal axis of the beam. Conforming transverse reinforcement consists of: (a) closed stirrups over the entire length of the beam at a spacing ≤ $d/3$, and (b) strength of closed stirrups $V_s \geq 3/4$ of required shear strength of beam.

Figure 6-1(a), only the longitudinal steel in the boundary of the wall should be included. If the wall does not have a boundary member, then only the longitudinal steel in the outer 25% of the wall section

Table 6-18 Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Members Controlled by Shear

Conditions		Drift Ratio (%), or Chord Rotation (radians)[1]		Residual Strength Ratio	Acceptable Drift (%) or Chord Rotation (radians)[1]				
					Component Type				
					Primary			Secondary	
					Performance Level				
		d	***e***	***c***	**IO**	**LS**	**CP**	**LS**	**CP**
i. Shear walls and wall segments									
All shear walls and wall segments[2]		0.75	2.0	0.40	0.40	0.60	0.75	0.75	1.5
ii. Shear wall coupling beams									
Longitudinal reinforcement and transverse reinforcement[3]	$\frac{\text{Shear}}{t_w l_w \sqrt{f_c'}}$								
Conventional longitudinal reinforcement with conforming transverse reinforcement	≤ 3	0.018	0.030	0.60	0.006	0.012	0.015	0.015	0.024
	≥ 6	0.012	0.020	0.30	0.004	0.008	0.010	0.010	0.016
Conventional longitudinal reinforcement with nonconforming transverse reinforcement	≤ 3	0.012	0.025	0.40	0.006	0.008	0.010	0.010	0.020
	≥ 6	0.008	0.014	0.20	0.004	0.006	0.007	0.007	0.012

1. For shear walls and wall segments, use drift; for coupling beams, use chord rotation; refer to Figures 6-3 and 6-4.
2. For shear walls and wall segments where inelastic behavior is governed by shear, the axial load on the member must be ≤ 0.15 $A_g f_c'$; otherwise, the member must be treated as a force-controlled component.
3. Conventional longitudinal reinforcement consists of top and bottom steel parallel to the longitudinal axis of the beam. Conforming transverse reinforcement consists of: (a) closed stirrups over the entire length of the beam at a spacing ≤ $d/3$, and (b) strength of closed stirrups V_s ≥ 3/4 of required shear strength of beam.

shall be included in the calculation of the yield strength. When calculating the nominal flexural strength of the wall, as represented by point *C* in Figure 6-1(a), all longitudinal steel (including web reinforcement) shall be included in the calculation. For both of the moment calculations described here, the yield strength of the longitudinal reinforcement should be taken as 125% of the specified yield strength to account for material overstrength and strain hardening. For all moment strength calculations, the axial load acting on the wall shall include gravity loads as defined in Chapter 3.

The nominal flexural strength of a shear wall or wall segment shall be used to determine the maximum shear force likely to act in shear walls, wall segments, and columns supporting discontinuous shear walls. For cantilever shear walls and columns supporting discontinuous shear walls, the design shear force is equal to the magnitude of the lateral force required to develop the nominal flexural strength at the base of the wall, assuming the lateral force is distributed uniformly over the height of the wall. For wall segments, the design shear force is equal to the shear corresponding to the development of the positive and negative nominal moment strengths at opposite ends of the wall segment.

The nominal shear strength of a shear wall or wall segment shall be determined based on the principles and equations given in Section 21.6 of *ACI 318-95*. The nominal shear strength of RC columns supporting discontinuous shear walls shall be determined based on the principles and equations given in Section 21.3 of *ACI 318-95*. For all shear strength calculations, 1.0 times the specified reinforcement yield strength should be used. There should be no difference between the yield and nominal shear strengths, as represented by points *B* and *C* in Figure 6-1.

When a shear wall or wall segment has a transverse reinforcement percentage, ρ_n, less than the minimum value of 0.0025 but greater than 0.0015, the shear strength of the wall shall be analyzed using the *ACI 318-95* equations noted above. For transverse reinforcement percentages less than 0.0015, the contribution from the wall reinforcement to the shear strength of the wall shall be held constant at the value obtained using ρ_n = 0.0015 (Wood, 1990).

Splice lengths for primary longitudinal reinforcement shall be evaluated using the procedures given in Section 6.4.5. Reduced flexural strengths shall be evaluated at locations where splices govern the usable stress in the reinforcement. The need for confinement reinforcement in shear wall boundary members shall be evaluated by the procedure in the *Uniform Building Code* (ICBO, 1994), or the method recommended by Wallace (1994 and 1995) for determining maximum lateral deformations in the wall and the resulting maximum compression strains in the wall boundary.

The nominal flexural and shear strengths of coupling beams reinforced with conventional reinforcement shall be evaluated using the principles and equations contained in Chapter 21 of *ACI 318-95*. The nominal flexural and shear strengths of coupling beams reinforced with diagonal reinforcement shall be evaluated using the procedure defined in the 1994 *NEHRP Recommended Provisions*. In both cases, 125% of the specified yield strength for the longitudinal and diagonal reinforcement should be used.

The nominal shear and flexural strengths of columns supporting discontinuous shear walls shall be evaluated as defined in Section 6.5.2.3.

6.8.2.4 Acceptance Criteria

A. Linear Static and Dynamic Procedures

All shear walls, wall segments, coupling beams, and columns supporting discontinuous shear walls shall be classified as either deformation- or force-controlled, as defined in Chapter 3. For columns supporting discontinuous shear walls, deformation-controlled actions shall be restricted to flexure. In the other components or elements noted here, deformation-controlled actions shall be restricted to flexure or shear. All other actions shall be defined as being force-controlled actions.

Design actions (flexure, shear, or force transfer at rebar anchorages and splices) on components shall be determined as prescribed in Chapter 3. When determining the appropriate value for the design actions, proper consideration should be given to gravity loads and to the maximum forces that can be transmitted considering nonlinear action in adjacent components. For example, the maximum shear at the base of a shear wall cannot exceed the shear required to develop the nominal flexural strength of the wall. Tables 6-19 and 6-20 present *m* values for use in Equation 3-18. Alternate *m* values are permitted where justified by experimental evidence and analysis.

B. Nonlinear Static and Dynamic Procedures

Inelastic response shall be restricted to those elements and actions listed in Tables 6-17 and 6-18, except where it is demonstrated that other inelastic actions can be tolerated considering the selected Performance Levels. For members experiencing inelastic behavior, the magnitude of other actions (forces, moments, or torque) in the member shall correspond to the magnitude of the action causing inelastic behavior. The magnitude of these other actions shall be shown to be below their nominal capacities.

For members experiencing inelastic response, the maximum plastic hinge rotations, drifts, or chord rotation angles shall not exceed the values given in Tables 6-17 and 6-18, for the particular Performance Level being evaluated. Linear interpolation between tabulated values shall be used if the member under analysis has conditions that are between the limits given in the tables. If the maximum plastic hinge rotation, drift, or chord rotation angle exceeds the corresponding value obtained either directly from the tables or by interpolation, the member shall be considered to be deficient, and either the member or the structure will need to be rehabilitated.

6.8.2.5 Rehabilitation Measures

All of the rehabilitation measures listed here for shear walls assume that a proper evaluation will be made of the wall foundation, diaphragms, and connections between existing structural elements and any elements added for rehabilitation purposes. Connection requirements are given in Section 6.4.6.

Table 6-19 Numerical Acceptance Criteria for Linear Procedures—Members Controlled by Flexure

Conditions			*m* factors				
			Component Type				
			Primary			Secondary	
			Performance Level				
			IO	LS	CP	LS	CP
i. Shear walls and wall segments							
$\frac{(A_s - A'_s)f_y + P}{t_w l_w f'_c}$	$\frac{\text{Shear}}{t_w l_w \sqrt{f'_c}}$	Confined Boundary[1]					
≤ 0.1	≤ 3	Yes	2	4	6	6	8
≤ 0.1	≥ 6	Yes	2	3	4	4	6
≥ 0.25	≤ 3	Yes	1.5	3	4	4	6
≥ 0.25	≥ 6	Yes	1	2	2.5	2.5	4
≤ 0.1	≤ 3	No	2	2.5	4	4	6
≤ 0.1	≥ 6	No	1.5	2	2.5	2.5	4
≥ 0.25	≤ 3	No	1	1.5	2	2	3
≥ 0.25	≥ 6	No	1	1	1.5	1.5	2
ii. Columns supporting discontinuous shear walls							
Transverse reinforcement[2]							
Conforming			1	1.5	2	n.a.	n.a.
Nonconforming			1	1	1	n.a.	n.a.
iii. Shear wall coupling beams							
Longitudinal reinforcement and transverse reinforcement[3]		$\frac{\text{Shear}}{t_w l_w \sqrt{f'_c}}$					
Conventional longitudinal reinforcement with conforming transverse reinforcement		≤ 3	2	4	6	6	9
		≥ 6	1.5	3	4	4	7
Conventional longitudinal reinforcement with nonconforming transverse reinforcement		≤ 3	1.5	3.5	5	5	8
		≥ 6	1.2	1.8	2.5	2.5	4
Diagonal reinforcement		n.a.	2	5	7	7	10

1. Requirements for a confined boundary are the same as those given in *ACI 318-95*.
2. Requirements for conforming transverse reinforcement are: (a) closed stirrups over the entire length of the column at a spacing ≤ $d/2$, and (b) strength of closed stirrups $V_s \geq$ required shear strength of column.
3. Conventional longitudinal reinforcement consists of top and bottom steel parallel to the longitudinal axis of the beam. Conforming transverse reinforcement consists of: (a) closed stirrups over the entire length of the beam at a spacing ≤ $d/3$, and (b) strength of closed stirrups $V_s \geq 3/4$ of required shear strength of beam.

Table 6-20 Numerical Acceptance Criteria for Linear Procedures—Members Controlled by Shear

Conditions		*m* factors				
		Component Type				
		Primary			Secondary	
		Performance Level				
		IO	LS	CP	LS	CP
i. Shear walls and wall segments						
All shear walls and wall segments[1]		2	2	3	2	3
ii. Shear wall coupling beams						
Longitudinal reinforcement and transverse reinforcement[2]	$\dfrac{\text{Shear}}{t_w l_w \sqrt{f_c'}}$					
Conventional longitudinal reinforcement with conforming transverse reinforcement	≤ 3	1.5	3	4	4	6
	≥ 6	1.2	2	2.5	2.5	3.5
Conventional longitudinal reinforcement with nonconforming transverse reinforcement	≤ 3	1.5	2.5	3	3	4
	≥ 6	1	1.2	1.5	1.5	2.5

1. For shear walls and wall segments where inelastic behavior is governed by shear, the axial load on the member must be $\leq 0.15 A_g f_c'$, the longitudinal reinforcement must be symmetrical, and the maximum shear stress must be $\leq 6\sqrt{f_c'}$, otherwise the shear shall be considered to be a force-controlled action.
2. Conventional longitudinal reinforcement consists of top and bottom steel parallel to the longitudinal axis of the beam. Conforming transverse reinforcement consists of: (a) closed stirrups over the entire length of the beam at a spacing $\leq d/3$, and (b) strength of closed stirrups $V_s \geq 3/4$ of required shear strength of beam.

- **Addition of wall boundary members.** Shear walls or wall segments that have insufficient flexural strength may be strengthened by the addition of boundary members. These members could be cast-in-place reinforced concrete elements or steel sections. In both cases, proper connections must be made between the existing wall and the added members. Also, the shear capacity of the rehabilitated wall will need to be reevaluated.

- **Addition of confinement jackets at wall boundaries.** The flexural deformation capacity of a shear wall can be improved by increasing the confinement at the wall boundaries. This is most easily achieved by the addition of a steel or reinforced concrete jacket. For both types of jackets, the longitudinal steel should not be continuous from story to story unless the jacket is also being used to increase the flexural capacity. The minimum thickness for a concrete jacket shall be three inches. Carbon fiber wrap may also be an effective method for improving the confinement of concrete in compression.

- **Reduction of flexural strength.** In some cases it may be desirable to reduce the flexural capacity of a shear wall to change the governing failure mode from shear to flexure. This is most easily accomplished by saw-cutting a specified number of longitudinal bars near the edges of the shear wall.

- **Increased shear strength of wall.** The shear strength provided by the web of a shear wall can be increased by casting additional reinforced concrete adjacent to the wall web. The new concrete should be at least four inches thick and should contain horizontal and vertical reinforcement. The new concrete will need to be properly bonded to the existing web of the shear wall. The use of carbon fiber sheets, epoxied to the concrete surface, can also increase the shear capacity of a shear wall.

- **Confinement jackets to improve deformation capacity of coupling beams and columns supporting discontinuous shear walls.** The use of confinement jackets has been discussed above for wall boundaries and in Section 6.5 for frame elements. The same procedures can be used to increase both the shear capacity and the deformation capacity of coupling beams and columns supporting discontinuous shear walls.

- **Infilling between columns supporting discontinuous shear walls.** Where a discontinuous shear wall is supported on columns that lack either sufficient strength or deformation capacity to satisfy design criteria, the opening between these columns may be infilled to make the wall continuous. The infill and existing columns should be designed to satisfy all the requirements for new wall construction. This may require strengthening of the existing columns by adding a concrete or steel jacket for strength and increased confinement. The opening below a discontinuous shear wall could also be “infilled” with steel bracing. The bracing members should be sized to satisfy all design requirements and the columns should be strengthened with a steel or a reinforced concrete jacket.

6.9 Precast Concrete Shear Walls

6.9.1 Types of Precast Shear Walls

Precast concrete shear walls typically consist of story-high or half-story-high precast wall segments that are made continuous through the use of either mechanical connectors or reinforcement splicing techniques, and, usually a cast-in-place connection strip. Connections between precast segments are typically made along both the horizontal and vertical edges of a wall segment. Tilt-up construction should be considered to be a special technique for precast wall construction. There are vertical joints between adjacent panels and horizontal joints at the foundation level and where the roof or floor diaphragm connects with the tilt-up panel.

If the reinforcement connections are made to be stronger than the adjacent precast panels, the lateral load response behavior of the precast wall system will be comparable to that for monolithic shear walls. This design approach is known as *cast-in-place emulation*.

An alternate design approach is to allow the inelastic action to occur at the connections between precast panels, an approach known as *jointed construction*. The provisions given here are intended for use with all types of precast wall systems.

6.9.1.1 Cast-In-Place Emulation

For this design approach, the connections between precast wall elements are designed and detailed to be stronger than the panels they connect. Thus, when the precast shear wall is subjected to lateral loading, any yielding and inelastic behavior should take place in the panel elements away from the connections. If the reinforcement detailing in the panel is similar to that for cast-in-place shear walls, then the inelastic response of a precast shear wall should be very similar to that for a cast-in-place wall.

Modern building codes permit the use of precast shear wall construction in high seismic zones if it satisfies the criteria for cast-in-place emulation. For such structures, the shear walls and wall segments can be evaluated by the criteria defined in Section 6.8.

6.9.1.2 Jointed Construction

For most older structures that contain precast shear walls, and for some modern construction, inelastic activity can be expected in the connections between precast wall panels during severe lateral loading. Because joints between precast shear walls in older buildings have often exhibited brittle behavior during inelastic load reversals, jointed construction had not been permitted in high seismic zones. Therefore, when evaluating older buildings that contain precast shear walls that are likely to respond as jointed construction, the permissible ductilities and rotation capacities given in Section 6.8 will have to be reduced.

For some modern structures, precast shear walls have been constructed with special connectors that are detailed to exhibit ductile response and energy absorption characteristics. Many of these connectors are proprietary and only limited experimental evidence concerning their inelastic behavior is available. Although this type of construction is clearly safer than jointed construction in older buildings, the experimental evidence is not sufficient to permit the use of the same ductility and rotation capacities given for cast-in-place construction. Thus, the permissible values given in Section 6.8 will need to be reduced.

6.9.1.3 Tilt-up Construction

Tilt-up construction should be considered to be a special case of jointed construction. The walls for most buildings constructed by the tilt-up method are longer than their height. Shear would usually govern their in-plane design, and their shear strength should be analyzed as force-controlled action. The major concern for most tilt-up construction is the connection between the tilt-up wall and the roof diaphragm. That connection should be carefully analyzed to be sure the diaphragm forces can be safely transmitted to the precast wall system.

6.9.2 Precast Concrete Shear Walls and Wall Segments

6.9.2.1 General Modeling Considerations

The analysis model for a precast concrete shear wall or wall segment shall represent the stiffness, strength, and deformation capacity of the overall member, as well as the connections and joints between any precast panel components that compose the wall. Potential failure in flexure, shear, and reinforcement development at any point in the shear wall panels or connections shall be considered. Interaction with other structural and nonstructural elements shall be included.

In most cases, precast concrete shear walls and wall segments within the precast panels may be modeled analytically as equivalent beam-columns that include both flexural and shear deformations. The rigid connection zone at beam connections to these equivalent beam-columns must properly represent the distance from the wall centroid—where the beam-column is placed—to the edge of the wall or wall segment. Unsymmetrical precast wall sections shall model the different bending capacities for the two loading directions.

For precast shear walls and wall segments where shear deformations will have a more significant effect on behavior, a multiple spring model should be used.

The diaphragm action of concrete slabs interconnecting precast shear walls and frame columns shall be properly represented.

6.9.2.2 Stiffness for Analysis

The modeling assumptions defined in Section 6.8.2.2 for monolithic concrete shear walls and wall segments shall also be used for precast concrete walls. In addition, the analytical model shall adequately model the stiffness of the connections between the precast components that compose the wall. This may be accomplished by softening the model used to represent the precast panels to account for flexibility in the connections. An alternative procedure would be to add spring elements to simulate axial, shear, and rotational deformations within the connections between panels.

A. Linear Static and Dynamic Procedures

The modeling procedures given in Section 6.8.2.2A, combined with a procedure for including connection deformations as noted above, shall be used.

B. Nonlinear Static and Dynamic Procedures

Nonlinear load-deformation relations shall follow the general guidelines of Section 6.4.1.2. The monotonic load-deformation relationships for analytical models that represent precast shear walls and wall elements within precast panels shall be represented by one of the general shapes defined in Figure 6-1. Values for plastic hinge rotations or drifts at points *B*, *C*, and *E* for the two general shapes are defined below. The strength levels at points *B* and *C* should correspond to the yield strength and nominal strength, as defined in Section 6.8.2.3. The residual strength for the line segment *D*–*E* is defined below.

For precast shear walls and wall segments whose inelastic behavior under lateral loading is governed by flexure, the general load-deformation relationship in Figure 6-1(a) will be referred to. For these members, the x-axis of Figure 6-1(a) should be taken as the rotation over the plastic hinging region at the end of the member (Figure 6-2). If the requirements for cast-in-place emulation are satisfied, the value of the hinge rotation at point *B* corresponds to the yield rotation, θ_y, and is given by Equation 6-5. The same expression should also be used for wall segments within a precast panel if flexure controls the inelastic response of the segment.

If the precast wall is of jointed construction and flexure governs the inelastic response of the member, then the value of θ_y will need to be increased to account for rotation in the joints between panels or between the panel and the foundation.

For precast shear walls and wall segments whose inelastic behavior under lateral loading is governed by shear, the general load-deformation relationship in

Figure 6-1(b) will be referred to. For these members, the x-axis of Figure 6-1(b) should be taken as the story drift for shear walls, and as the element drift for wall segments (Figure 6-3).

For construction classified as cast-in-place emulation, the values for the variables *a*, *b*, and *c*, which are required to define the location of points *C*, *D*, and *E* in Figure 6-1(a), are given in Table 6-17. For construction classified as jointed construction, the values of *a*, *b*, and *c* given in Table 6-17 shall be reduced to 50% of the given values, unless there is experimental evidence available to justify higher values. In no case, however, shall values larger than those given in Table 6-17 be used.

For construction classified as cast-in-place emulation, values for the variables *d*, *e*, and *c*, which are required to find the points *C*, *D*, and *E* in Figure 6-1(b), are given in Tables 6-17 and 6-18 for the appropriate member conditions. For construction classified as jointed construction, the values of *d*, *e*, and *c* given in Tables 6-17 and 6-18 shall be reduced to 50% of the given values unless there is experimental evidence available to justify higher values. In no case, however, shall values larger than those given in Tables 6-17 and 6-18 be used.

For Tables 6-17 and 6-18, linear interpolation between tabulated values shall be used if the member under analysis has conditions that are between the limits given in the tables.

For the NDP, the complete hysteresis behavior of each component shall be modeled using properties verified by experimental evidence. The relationships in Figure 6-1 may be taken to represent the envelope for the analysis. The unloading and reloading stiffnesses and strengths, and any pinching of the load versus rotation hysteresis loops, shall reflect the behavior experimentally observed for wall elements similar to the one under investigation.

6.9.2.3 Design Strengths

The strength of precast concrete shear walls and wall segments within the panels shall be computed according to the general requirement of Section 6.4.2, except as modified here. For cast-in-place emulation types of construction, the strength calculation procedures given in Section 6.8.2.3 shall be followed.

For jointed construction, calculations of axial, shear, and flexural strength of the connections between panels shall be based on known or assumed material properties and the fundamental principles of structural mechanics. Yield strength for steel reinforcement of connection hardware used in the connections shall be increased to 125% of its specified yield value when calculating the axial and flexural strength of the connection region. The unmodified specified yield strength of the reinforcement and connection hardware shall be used when calculating the shear strength of the connection region.

In older construction, particular attention must be given to the technique used for splicing reinforcement extending from adjacent panels into the connection. These connections may be insufficient and can often govern the strength of the precast shear wall system. If sufficient detail is not given on the design drawings, concrete should be removed in some connections to expose the splicing details for the reinforcement.

For all precast concrete shear walls of jointed construction, no difference shall be taken between the computed yield and nominal strengths in flexure and shear. Thus, the values for strength represented by the points *B* and *C* in Figure 6-1 shall be computed following the procedures given in Section 6.8.2.3.

6.9.2.4 Acceptance Criteria

A. Linear Static and Dynamic Procedures

For precast shear wall construction that emulates cast-in-place construction and for wall segments within a precast panel, the acceptance criteria defined in Section 6.8.2.4A shall be followed. For precast shear wall construction defined as jointed construction, the acceptance criteria procedure given in Section 6.8.2.4A shall be followed. However, the *m* values given in Tables 6-19 and 6-20 shall be reduced by 50%, unless experimental evidence justifies the use of a larger value. In no case shall an *m* value be taken as less than 1.0.

B. Nonlinear Static and Dynamic Procedures

Inelastic response shall be restricted to those shear walls (and wall segments) and actions listed in Tables 6-17 and 6-18, except where it is demonstrated that other inelastic action can be tolerated considering the selected Performance Levels. For members experiencing inelastic behavior, the magnitude of the other actions (forces, moments, or torques) in the member shall correspond to the magnitude of the action causing the inelastic behavior. The magnitude of these other actions shall be shown to be below their nominal capacities.

For precast shear walls of the cast-in-place emulation type of construction, and for wall segments within a precast panel, the maximum plastic hinge rotation angles or drifts during inelastic response shall not exceed the values given in Tables 6-17 and 6-18. For precast shear walls of jointed construction, the maximum plastic hinge rotation angles or drifts during inelastic response shall not exceed one-half of the values given in Tables 6-17 and 6-18, unless experimental evidence is available to justify a higher value. However, in no case shall deformation values larger than those given in these tables be used for jointed type construction.

If the maximum deformation value exceeds the corresponding tabular value, the element shall be considered to be deficient and either the element or structure will need to be rehabilitated.

6.9.2.5 Rehabilitation Measures

Precast concrete shear wall systems may suffer from some of the same deficiencies as cast-in-place walls. These may include inadequate flexural capacity, inadequate shear capacity with respect to flexural capacity, lack of confinement at wall boundaries, and inadequate splice lengths for longitudinal reinforcement in wall boundaries. All of these deficiencies can be rehabilitated by the use of one of the measures described in Section 6.8.2.5. A few deficiencies unique to precast wall construction are inadequate connections between panels, to the foundation, and to floor or roof diaphragms.

- **Enhancement of connections between adjacent or intersecting precast wall panels.** A combination of mechanical and cast-in-place details may be used to strengthen connections between precast panels. Mechanical connectors may include steel shapes and various types of drilled-in anchors. Cast-in-place strengthening methods generally involve exposing the reinforcing steel at the edges of adjacent panels, adding vertical and transverse (tie) reinforcement, and placing new concrete.

- **Enhancement of connections between precast wall panels and foundations.** The shear capacity of the wall panel-to-foundation connection can be strengthened by the use of supplemental mechanical connectors or by using a cast-in-place overlay with new dowels into the foundation. The overturning moment capacity of the panel-to-foundation connection can be strengthened by using drilled-in dowels within a new cast-in-place connection at the edges of the panel. Adding connections to adjacent panels may also eliminate some of the forces transmitted through the panel-to-foundation connection.

- **Enhancement of connections between precast wall panels and floor or roof diaphragms.** These connections can be strengthened by using either supplemental mechanical devices or cast-in-place connectors. Both in-plane shear and out-of-plane forces will need to be considered when strengthening these connections.

6.10 Concrete Braced Frames

6.10.1 Types of Concrete Braced Frames

Reinforced concrete braced frames are those frames with monolithic reinforced concrete beams, columns, and diagonal braces that are coincident at beam-column joints. Components are nonprestressed. Under lateral loading, the braced frame resists loads primarily through truss action.

Masonry infills may be present in braced frames. Where masonry infills are present, requirements for masonry infilled frames as specified in Section 6.7 also apply.

The provisions are applicable to existing reinforced concrete braced frames, and existing reinforced concrete braced frames rehabilitated by addition or removal of material.

6.10.2 General Considerations in Analysis and Modeling

The analysis model for a reinforced concrete braced frame shall represent the strength, stiffness, and deformation capacity of beams, columns, braces, and all connections and components that may be part of the element. Potential failure in tension, compression (including instability), flexure, shear, anchorage, and reinforcement development at any section along the component length shall be considered. Interaction with other structural and nonstructural elements and components shall be included.

The analytical model generally can represent the framing, using line elements with properties concentrated at component centerlines. General considerations relative to the analytical model are summarized in Section 6.5.2.1.

In frames having braces in some bays and no braces in other bays, the restraint of the brace shall be represented as described above, and the nonbraced bays shall be modeled as frames according to the specifications of this chapter. Where braces create a vertically discontinuous frame, the effects on overall building performance shall be considered.

Inelastic deformations in primary components shall be restricted to flexure and axial load in beams, columns, and braces. Other inelastic deformations are permitted in secondary components. Acceptance criteria are presented in Section 6.10.5.

6.10.3 Stiffness for Analysis

6.10.3.1 Linear Static and Dynamic Procedures

Beams, columns, and braces in braced portions of the frame may be modeled considering axial tension and compression flexibilities only. Nonbraced portions of frames shall be modeled according to procedures described elsewhere for frames. Effective stiffnesses shall be according to Section 6.4.1.2.

6.10.3.2 Nonlinear Static Procedure

Nonlinear load-deformation relations shall follow the general guidelines of Section 6.4.1.2.

Beams, columns, and braces in braced portions may be modeled using nonlinear truss components. Beams and columns in nonbraced portions may be modeled using procedures described elsewhere in this chapter. The model shall be capable of representing inelastic response along the component lengths, as well as within connections.

Numerical quantities in Figure 6-1 may be derived from tests or rational analyses. Alternately, the guidelines of Section 6.7.2.2B may be used, with braces modeled as columns per Table 6-15.

6.10.3.3 Nonlinear Dynamic Procedure

For the NDP, the complete hysteresis behavior of each component shall be modeled using properties verified by tests. Unloading and reloading properties shall represent significant stiffness and strength degradation characteristics.

6.10.4 Design Strengths

Component strengths shall be computed according to the general requirements of Section 6.4.2 and the additional requirements of Section 6.5.2.3. The possibility of instability of braces in compression shall be considered.

6.10.5 Acceptance Criteria

6.10.5.1 Linear Static and Dynamic Procedures

All component actions shall be classified as being either deformation-controlled or force-controlled, as defined in Chapter 3. In primary components, deformation-controlled actions shall be restricted to flexure and axial actions in beams and columns, and axial actions in braces. In secondary components, deformation-controlled actions shall be restricted to those actions identified for the braced or isolated frame in this chapter.

Calculated component actions shall satisfy the requirements of Chapter 3. Refer to other sections of this chapter for *m* values for concrete frames, except that *m* values for beams, columns, and braces modeled as tension and compression components may be taken as equal to values specified for columns in Table 6-16. Values of *m* shall be reduced from values in that table where component buckling is a consideration. Alternate approaches or values are permitted where justified by experimental evidence and analysis.

6.10.5.2 Nonlinear Static and Dynamic Procedures

Calculated component actions shall not exceed the numerical values listed in Table 6-15 or the relevant tables for isolated frames given elsewhere in this chapter. Where inelastic action is indicated for a component or action not listed in these tables, the performance shall be deemed unacceptable. Alternate approaches or values are permitted where justified by experimental evidence and analysis.

6.10.6 Rehabilitation Measures

Rehabilitation measures include the general approaches listed for other elements in this chapter, plus other approaches based on rational procedures.

Rehabilitated frames shall be evaluated according to the general principles and requirements of this chapter. The effects of rehabilitation on stiffness, strength, and deformability shall be taken into account in the analytical model. Connections required between existing and new elements shall satisfy requirements of Section 6.4.6 and other requirements of the *Guidelines*.

6.11 Concrete Diaphragms

6.11.1 Components of Concrete Diaphragms

Cast-in-place concrete diaphragms transmit inertial forces from one location in a structure to a vertical lateral-force-resisting element. A concrete diaphragm is generally a floor or roof slab, but can be a structural truss in the horizontal plane.

Diaphragms are made up of slabs that transmit shear forces, struts that provide continuity around openings, collectors that gather force and distribute it, and chords that are located at the edges of diaphragms and that resist tension and compression forces.

6.11.1.1 Slabs

The primary function of any slab that is part of a floor or roof system is to support gravity loads. A slab must also function as part of the diaphragm to transmit the shear forces associated with the load transfer. These internal shear forces are generated when the slab is the load path for forces that are being transmitted from one vertical lateral-force-resisting system to another, or when the slab is functioning to provide bracing to other portions of the building that are being loaded out of plane. Included in this section are all versions of cast-in-place concrete floor systems, and concrete-on-metal deck systems.

6.11.1.2 Struts and Collectors

Struts and collectors are built into diaphragms in locations where there are defined stress demands that exceed the typical stress capacity of the diaphragm. These locations occur around openings in the diaphragms, along defined load paths between lateral-load-resisting elements, and at intersections of portions of floors that have plan irregularities. Struts and collectors may occur within the slab thickness or may have the form of cast-in-place beams that are monolithic with the slabs. The forces that they resist are primarily axial in nature, but may also include shear and bending forces.

6.11.1.3 Diaphragm Chords

Diaphragm chords generally occur at the edges of a horizontal diaphragm and function to resist bending stresses in the diaphragm. Tensile forces typically are most critical, but compressive forces in thin slabs could be a problem. Exterior walls can serve this function if there is adequate horizontal shear capacity between the slab and wall. When evaluating an existing building, special care should be taken to evaluate the condition of the lap splices. Where the splices are not confined by closely-spaced transverse reinforcement, splice failure is possible if stress levels reach critical values. In rehabilitation construction, new laps should be confined by closely-spaced transverse reinforcement.

6.11.2 Analysis, Modeling, and Acceptance Criteria

6.11.2.1 General Considerations

The analysis model for a diaphragm shall represent the strength, stiffness, and deformation capacity of each component and the diaphragm as a whole. Potential failure in flexure, shear, buckling, and reinforcement development at any point in the diaphragm shall be considered.

The analytical model of the diaphragm can typically be taken as a continuous or simple span horizontal beam that is supported by elements of varying stiffness. The beam may be rigid or semi-rigid. Most computer models assume a rigid diaphragm. Few cast-in-place diaphragms would be considered flexible, whereas a thin concrete slab on a metal deck might be semi-rigid depending on the length-to-width ratio of the diaphragm.

6.11.2.2 Stiffness for Analysis

Diaphragm stiffness shall be modeled according to Section 6.11.2.1 and shall be determined using a linear elastic model and gross section properties. The modulus of elasticity used shall be that of the concrete as specified in Section 8.5.1 of *ACI 318-95*. When the length-to-width ratio of the diaphragm exceeds 2.0 (where the length is the distance between vertical elements), the effects of diaphragm deflection shall be considered when assigning lateral forces to the resisting vertical elements. The concern is for relatively flexible vertical members that may be displaced by the diaphragm, and for relatively stiff vertical members that may be overloaded due to the same diaphragm displacement.

6.11.2.3 Design Strengths

Component strengths shall be according to the general requirements of Section 6.4.2, as modified in this section.

The maximum component strength shall be determined considering potential failure in flexure, axial load, shear, torsion, development, and other actions at all points in the component under the actions of design gravity and lateral load combinations. The shear strength shall be as specified in Section 21.6.4 of *ACI 318-95*. Strut, collector, and chord strengths shall be determined according to Section 6.5.2.3 of these *Guidelines*.

6.11.2.4 Acceptance Criteria

All component actions shall be classified as either deformation-controlled or force-controlled, as defined in Chapter 3. Diaphragm shear shall be considered as being a force-controlled component and shall have a DCR not greater than 1.25. Acceptance criteria for all other component actions shall be as defined in Section 6.5.2.4A, with *m* values taken according to similar components in Tables 6-10 and 6-11 for use in Equation 3-18. Analysis shall be restricted to linear procedures.

6.11.3 Rehabilitation Measures

Cast-in-place concrete diaphragms can have a wide variety of deficiencies; see Chapter 10 and FEMA 178 (BSSC, 1992a). Two general alternatives may be used to correct deficiencies: either improving the strength and ductility, or reducing the demand in accordance with FEMA 172 (BSSC, 1992b). Individual components can be strengthened or improved by adding additional reinforcement and encasement. Diaphragm thickness may be increased, but the added weight may overload the footings and increase the seismic load. Demand can be lowered by adding additional lateral-force-resisting elements, introducing additional damping, or base isolating the structure. All corrective measures taken shall be based upon engineering mechanics, taking into account load paths and deformation compatibility requirements of the structure.

6.12 Precast Concrete Diaphragms

6.12.1 Components of Precast Concrete Diaphragms

Section 6.11 provided a general overview of concrete diaphragms. Components of precast concrete diaphragms are similar in nature and function to those of cast-in-place diaphragms, with a few critical differences. One is that precast diaphragms do not possess the inherent unity of cast-in-place monolithic construction. Additionally, precast components may be highly stressed due to prestressed forces. These forces cause long-term shrinkage and creep, which shorten the component over time. This shortening tends to fracture connections that restrain the component.

Precast concrete diaphragms can be classified as topped or untopped. A topped diaphragm is one that has had a concrete topping slab poured over the completed horizontal system. Most floor systems have a topping system, but some hollow core floor systems do not. The topping slab generally bonds to the top of the precast elements, but may have an inadequate thickness at the center of the span, or may be inadequately reinforced. Also, extensive cracking of joints may be present along the panel joints. Shear transfer at the edges of precast concrete diaphragms is especially critical.

Some precast roof systems are constructed as untopped systems. Untopped precast concrete diaphragms have been limited to lower seismic zones by recent versions of the *Uniform Building Code*. This limitation has been imposed because of the brittleness of connections and lack of test data concerning the various precast systems. Special consideration shall be given to diaphragm chords in precast construction.

6.12.2 Analysis, Modeling, and Acceptance Criteria

Analysis and modeling of precast concrete diaphragms shall conform to Section 6.11.2.2, with the added requirement that special attention be paid to considering the segmental nature of the individual components.

Component strengths shall be determined according to Section 6.11.2.3, with the following exception. Welded connection strength shall be determined using the latest version of the Precast Concrete Institute (PCI) Handbook, assuming that the connections have little ductility unless test data are available to document the assumed ductility.

Acceptance criteria shall be as defined in Section 6.11.2.4; the criteria of Section 6.4.6.2, where applicable, shall also be included.

6.12.3 Rehabilitation Measures

Section 6.11.3 provides guidance for rehabilitation measures for concrete diaphragms in general. Special care shall be taken to overcome the segmental nature of precast concrete diaphragms, and to avoid fracturing prestressing strands when adding connections.

6.13 Concrete Foundation Elements

6.13.1 Types of Concrete Foundations

Foundations serve to transmit loads from the vertical structural subsystems (columns and walls) of a building to the supporting soil or rock. Concrete foundations for buildings are classified as either shallow or deep foundations. Shallow foundations include spread or isolated footings; strip or line footings; combination footings; and concrete mat footings. Deep foundations include pile foundations and cast-in-place piers. Concrete grade beams may be present in both shallow and deep foundation systems.

These provisions are applicable to existing foundation elements and to new materials or elements that are required to rehabilitate an existing building.

6.13.1.1 Shallow Foundations

Existing spread footings, strip footings, and combination footings may be reinforced or unreinforced. Vertical loads are transmitted to the soil by direct bearing; lateral loads are transmitted by a combination of friction between the bottom of the footing and the soil, and passive pressure of the soil on the vertical face of the footing.

Concrete mat footings must be reinforced to resist the flexural and shear stresses resulting from the superimposed concentrated and line structural loads and the distributed resisting soil pressure under the footing. Lateral loads are resisted primarily by friction between the soil and the bottom of the footing, and by passive pressure developed against foundation walls that are part of the system.

6.13.1.2 Deep Foundations

A. Driven Pile Foundations

Concrete pile foundations are composed of a reinforced concrete pile cap supported on driven piles. The piles may be concrete (with or without prestressing), steel shapes, steel pipes, or composite (concrete in a driven steel shell). Vertical loads are transmitted to the piling by the pile cap, and are resisted by direct bearing of the pile tip in the soil or by skin friction or cohesion of the soil on the surface area of the pile. Lateral loads are resisted by passive pressure of the soil on the vertical face of the pile cap, in combination with interaction of the piles in bending and passive soil pressure on the pile surface. In poor soils, or soils subject to liquefaction, bending of the piles may be the only dependable resistance to lateral loads.

B. Cast-in-Place Pile Foundations

Cast-in-place concrete pile foundations consist of reinforced concrete placed in a drilled or excavated shaft. The shaft may be formed or bare. Segmented steel cylindrical liners are available to form the shaft in weak soils and allow the liner to be removed as the concrete is placed. Various slurry mixes are often used to protect the drilled shaft from caving soils; the slurry is then displaced as the concrete is placed by the tremie method. Cast-in-place pile or pier foundations resist vertical and lateral loads in a manner similar to that of driven pile foundations.

6.13.2 Analysis of Existing Foundations

The analytical model for concrete buildings, with columns or walls cast monolithically with the foundation, is sometimes assumed to have the vertical structural elements fixed at the top of the foundation. When this is assumed, the foundations and supporting soil must be capable of resisting the induced moments. When columns are not monolithic with their foundations, or are designed so as to not resist flexural moments, they may be modeled with pinned ends. In such cases, the column base must be evaluated for the resulting axial and shear forces as well as the ability to accommodate the necessary end rotation of the columns. The effects of base fixity of columns must be taken into account at the point of maximum displacement of the superstructure.

Overturning moments and economics may dictate the use of more rigorous Analysis Procedures. When this is the case, appropriate vertical, lateral, and rotational soil springs shall be incorporated in the analytical model as described in Section 4.4.2. The spring characteristics shall be based on the material in Chapter 4, and on the recommendations of the geotechnical consultant. Rigorous analysis of structures with deep foundations in soft soils will require special soil/pile interaction studies to determine the probable location of the point of fixity in the foundation and the resulting distribution of forces and displacements in the superstructure. In these analyses, the appropriate representation of the connection of the pile to the pile cap is required. Buildings designed for gravity loads only may have a nominal (about six inches) embedment of the piles without any dowels into the pile cap. These piles must be modeled as being "pinned" to the cap. Unless the connection can be identified from the available construction documents, the "pinned" connection should be assumed in any analytical model.

When the foundations are included in the analytical model, the responses of the foundation components may be derived by any of the analytical methods prescribed in Chapter 3, as modified by the requirements of Section 6.4. When the structural elements of the analytical model are assumed to be pinned or fixed at the foundation level, the reactions (axial loads, shears, and moments) of those elements shall be used to evaluate the individual components of the foundation system.

6.13.3 Evaluation of Existing Condition

Allowable soil capacities (subgrade modulus, bearing pressure, passive pressure) are a function of the chosen Performance Level, and will be as prescribed in Chapter 4 or as established with project-specific data by a geotechnical consultant. All components of existing foundation elements, and all new material, components, or elements required for rehabilitation, will be considered to be force-controlled ($m = 1.0$) based on the mechanical and analytical properties in Section 6.3.3. However, the capacity of the foundation components need not exceed 1.25 times the capacity of the supported vertical structural component or element (column or wall). The amount of foundation displacement that is acceptable for the given structure should be determined by the design engineer, and is a function of the desired Performance Level.

6.13.4 Rehabilitation Measures

The following general rehabilitation measures are applicable to existing foundation elements. Other approaches, based on rational procedures, may also be utilized.

6.13.4.1 Rehabilitation Measures for Shallow Foundations

- **Enlarging the existing footing by lateral additions**. The existing footing will continue to resist the loads and moment acting at the time of rehabilitation (unless temporarily removed). The enlarged footing is to resist subsequent loads and moments produced by earthquakes if the lateral additions are properly tied into the existing footing. Shear transfer and moment development must be accomplished in the additions.

- **Underpinning the footing.** Underpinning involves the removal of unsuitable soil under an existing footing, coupled with replacement using concrete, soil cement, suitable soil, or other material, and must be properly staged in small increments so as not to endanger the stability of the structure. This technique also serves to enlarge an existing footing or to extend it to a more competent soil stratum.

- **Providing tension hold-downs.** Tension ties (soil and rock anchors—prestressed and unstressed) are drilled and grouted into competent soils and anchored in the existing footing to resist uplift. Increased soil bearing pressures produced by the ties must be checked against values associated with the desired Performance Level. Piles or drilled piers may also be utilized.

- **Increasing effective depth of footing.** This method involves pouring new concrete to increase shear and moment capacity of existing footing. New horizontal reinforcement can be provided, if required, to resist increased moments.

- **Increasing the effective depth of a concrete mat foundation with a reinforced concrete overlay.** This method involves pouring an integral topping slab over the existing mat to increase shear and moment capacity. The practicality must be checked against possible severe architectural restrictions.

- **Providing pile supports for concrete footings or mat foundations.** Addition of piles requires careful design of pile length and spacing to avoid overstressing the existing foundations. The technique may only be feasible in a limited number of cases for driven piles, but special augered systems have been developed and are used regularly.

- **Changing the building structure to reduce the demand on the existing elements.** This method involves removing mass or height of the building or adding other materials or components (such as energy dissipation devices) to reduce the load transfer at the base level. The addition of new shear walls or braces will generally reduce the demand on existing foundations.

- **Adding new grade beams.** Grade beams may be used to tie existing footings together when poor soil exists, to provide fixity to column bases, and to distribute lateral loads between individual footings, pile caps, or foundation walls.

- **Improving existing soil.** Grouting techniques may be used to improve existing soil.

6.13.4.2 Rehabilitation Measures for Deep Foundations

- **Providing additional piles or piers.** The addition of piles or piers may require extension and additional reinforcement of existing pile caps. See the comments in previous sections for extending an existing footing.

- **Increasing the effective depth of the pile cap.** Addition of new concrete and reinforcement to the top of the cap is done to increase shear and moment capacity.

- **Improving soil adjacent to existing pile cap.** See Section 4.6.1.

- **Increasing passive pressure bearing area of pile cap.** Addition of new reinforced concrete extensions to the existing pile cap provides more vertical foundation faces and greater load transferability.

- **Changing the building system to reduce the demands on the existing elements.** Introduction of new lateral-load-resisting elements may reduce demand.

- **Adding batter piles or piers.** Batter piles or piers may be used to resist lateral loads. It should be noted that batter piles have performed poorly in recent earthquakes when liquefiable soils were present. This is especially important to consider around wharf structures and in areas that have a high water table. See Sections 4.2.2.2, 4.3.2, and 4.4.2.2B.

- **Increasing tension tie capacity from pile or pier to superstructure.**

6.14 Definitions

The definitions used in this chapter generally follow those of BSSC (1995) as well as those published in ACI 318. Many of the definitions that are independent of material type are provided in Chapter 2.

6.15 Symbols

A_g	Gross area of column, in.^{2}
A_j	Effective cross-sectional area within a joint, in.^{2}, in a plane parallel to plane of reinforcement generating shear in the joint. The joint depth shall be the overall depth of the column. Where a beam frames into a support of larger width, the effective width of the joint shall not exceed the smaller of: (1) beam width plus the joint depth, and (2) twice the smaller perpendicular distance from the longitudinal axis of the beam to the column side.
A_s	Area of nonprestressed tension reinforcement, in.^{2}
A'_s	Area of compression reinforcement, in.^{2}
A_w	Area of the web cross section, $= b_w d$
E_c	Modulus of elasticity of concrete, psi
E_s	Modulus of elasticity of reinforcement, psi
I	Moment of inertia
I_g	Moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement
L	Length of member along which deformations are assumed to occur
M_{gCS}	Moment acting on the slab column strip according to ACI 318

M_n Nominal moment strength at section

M_{nCS} Nominal moment strength of the slab column strip

M_y Yield moment strength at section

N_u Factored axial load normal to cross section occurring simultaneously with V_u. To be taken as positive for compression, negative for tension, and to include effects of tension due to creep and shrinkage.

P Axial force in a member, lbs

P_o Nominal axial load strength at zero eccentricity

Q Generalized load

Q_{CE} Expected strength of a component or element at the deformation level under consideration for deformation-controlled actions

Q_{CL} Lower-bound estimate of the strength of a component or element at the deformation level under consideration for force-controlled actions

V Design shear force at section

V_c Nominal shear strength provided by concrete

V_g Shear acting on slab critical section due to gravity loads

V_n Nominal shear strength at section

V_o Shear strength of slab at critical section

V_s Nominal shear strength provided by shear reinforcement

V_u Factored shear force at section

a Parameter used to measure deformation capacity

b Parameter used to measure deformation capacity

b_w Web width, in.

c Parameter used to measure residual strength

c_1 Size of rectangular or equivalent rectangular column, capital, or bracket measured in the direction of the span for which moments are being determined, in.

d Parameter used to measure deformation capacity

d Distance from extreme compression fiber to centroid of tension reinforcement, in.

d_b Nominal diameter of bar, in.

e Parameter used to measure deformation capacity

f'_c Compressive strength of concrete, psi

f_{pc} Average compressive stress in concrete due to effective prestress force only (after allowance for all prestress losses)

f_s Stress in reinforcement, psi

f_y Yield strength of tension reinforcement

h Height of member along which deformations are measured

h Overall thickness of member, in.

h_c Gross cross-sectional dimension of column core measured in the direction of joint shear, in.

h_w Total height of wall from base to top, in.

k Coefficient used for calculation of column shear strength

l_b Provided length of straight development, lap splice, or standard hook, in.

l_d Development length for a straight bar, in.

l_e Length of embedment of reinforcement, in.

l_p Length of plastic hinge used for calculation of inelastic deformation capacity, in.

l_w Length of entire wall or a segment of wall considered in the direction of shear force, in.

m Modification factor used in the acceptance criteria of deformation-controlled components or elements, indicating the available ductility of a component action

t_w Thickness of wall web, in.

Δ Generalized deformation, consistent units

γ Coefficient for calculation of joint shear strength

γ_f Fraction of unbalanced moment transferred by flexure at slab-column connections

θ Generalized deformation, radians

θ_y Yield rotation, radians

κ A reliability coefficient used to reduce component strength values for existing components, based on the quality of knowledge about the components' properties (see Section 2.7.2)

λ	Correction factor related to unit weight of concrete
μ	Coefficient of friction
ρ	Ratio of nonprestressed tension reinforcement
ρ'	Ratio of nonprestressed compression reinforcement
ρ''	Reinforcement ratio for transverse joint reinforcement
ρ_{bal}	Reinforcement ratio producing balanced strain conditions
ρ_n	Ratio of distributed shear reinforcement in a plane perpendicular to the direction of the applied shear

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7. Masonry (Systematic Rehabilitation)

7.1 Scope

This chapter describes engineering procedures for estimating seismic performance of vertical lateral-force-resisting masonry elements. The methods are applicable for masonry wall and infill panels that are either existing or rehabilitated elements of a building system, or new elements that are added to an existing building system.

This chapter presents the information needed for systematic rehabilitation of masonry buildings as depicted in Step 3 of the Process Flow chart shown in Figure 1-1. A brief historical perspective is given in Section 7.2, with an expanded version in *Commentary* Section C7.2. Masonry material properties for new and existing construction are discussed in Section 7.3.

Attributes of masonry walls and masonry infills are given in Sections 7.4 and 7.5, respectively. Masonry components are classified by their behavior; unreinforced components precede reinforced components, and in-plane action is separated from out-of-plane action. For each component type, the information needed to model stiffness is presented first, followed by recommended strength and deformation acceptance criteria for various performance levels. These attributes are presented in a format for direct use with the Linear and Nonlinear Static Procedures prescribed in Chapter 3. Guidelines for anchorage to masonry walls and masonry foundation elements are given in Sections 7.6 and 7.7, respectively. Section 7.8 provides definitions for terms used in this chapter, and Section 7.9 lists the symbols used in Chapter 7 equations. Applicable reference standards are listed in Section 7.10.

Portions of a masonry building that are not subject to systematic rehabilitation provisions of this chapter—such as parapets, cladding, or partition walls—shall be considered with the Simplified Rehabilitation options of Chapter 10 or with the provisions for nonstructural components addressed in Chapter 11.

The provisions of this chapter are intended for solid or hollow clay-unit masonry, solid or hollow concrete-unit masonry, and hollow clay tile. Stone or glass block masonry is not covered in this chapter.

The properties and behavior of steel, concrete, and timber floor or roof diaphragms are addressed in Chapters 5, 6, and 8, respectively. Connections to masonry walls are addressed in Section 7.6 for cases where behavior of the connection is dependent on properties of the masonry. Attributes for masonry foundation elements are briefly described in Section 7.7.

Unreinforced masonry buildings with flexible floor diaphragms may be evaluated by using the procedures given in Appendix C of FEMA 178 (BSSC, 1992) if the simplified rehabilitation approach of Chapter 10 is followed.

7.2 Historical Perspective

Construction of existing masonry buildings in the United States dates back to the 1500s in the southeastern and southwestern parts of the country, to the 1770s in the central and eastern parts, and to the 1850s in the western half of the nation. The stock of existing masonry buildings in the United States largely comprises structures constructed in the last 150 years. Since the types of units, mortars, and construction methods have changed over this course of time, knowing the vintage of a masonry building may be useful in identifying the characteristics of the construction. Although structural properties cannot be inferred solely from age, some background on typical materials and methods for a given period can help to improve engineering judgment, and provide some direction in the assessment of an existing building.

As indicated in Chapter 1, great care should be exercised in selecting the appropriate rehabilitation approaches and techniques for application to historic buildings in order to preserve their unique characteristics.

Section C7.2 of the *Commentary* provides an extensive historical perspective on various masonry materials and construction practices.

7.3 Material Properties and Condition Assessment

7.3.1 General

The methods specified in Section 7.3.2 for determination of mechanical properties of existing masonry construction shall be used as the basis for stiffness and strength attributes of masonry walls and infill panels, along with the methods described in Sections 7.4 and 7.5. Properties of new masonry components that are added to an existing structural system shall be based on values given in BSSC (1995).

Minimum requirements for determining in situ masonry compressive, tensile, and shear strength, as well as elastic and shear moduli, are provided in Section 7.3.2. Recommended procedures for measurement of each material property are described in corresponding sections of the *Commentary*. Data from testing of in-place materials shall be expressed in terms of mean values for determination of expected component strengths, Q_{CE}, and lower bound strengths, Q_{CL}, with the linear or nonlinear procedures described in Chapter 3.

In lieu of in situ testing, default values of material strength and modulus, as given in Section 7.3.2, shall be assigned to masonry components in good, fair, and poor condition. Default values represent typical lower bound estimates of strength or stiffness for all masonry nationwide, and thus should not be construed as expected values for a specific structure. As specified in Section 7.3.4, certain in situ tests are needed to attain the comprehensive level of knowledge (a κ value of 1.00) needed in order to use the nonlinear procedures of Chapter 3. Thus, unless noted otherwise, general use of the default values without in situ testing is limited to the linear procedures of Chapter 3.

Procedures for defining masonry structural systems, and assessing masonry condition, shall be conducted in accordance with provisions stated in Section 7.3.3. Requirements for either a minimum or a comprehensive level of evaluation, as generally stated in Section 2.7, are further refined for masonry components in Section 7.3.4.

7.3.2 Properties of In-Place Materials

7.3.2.1 Masonry Compressive Strength

Expected masonry compressive strength, f_{me}, shall be measured using one of the following three methods.

1. Test prisms shall be extracted from an existing wall and tested per Section 1.4.B.3 of the Masonry Standards Joint Committee's *Building Code Requirements for Masonry Structures* (MSJC, 1995a).

2. Prisms shall be fabricated from actual extracted masonry units, and a surrogate mortar designed on the basis of a chemical analysis of actual mortar samples. The test prisms shall be tested per Section 1.4.B.3 of the *Specification for Masonry Structures* (MSJC, 1995b).

3. Two flat jacks shall be inserted into slots cut into mortar bed joints and pressurized until peak stress is reached.

For each of the three methods, the expected compressive strength shall be based on the net mortared area.

If the masonry unit strength and the mortar type are known, f_{me} values may be taken from Tables 1 and 2 of MSJC (1995a) for clay or concrete masonry constructed after 1960. The f_{me} value shall be obtained by multiplying the table values by a factor that represents both the ratio of expected to lower bound strength and the height-to-thickness ratio of the prism (see *Commentary* Section C7.3.2.1).

In lieu of material tests, default values for masonry prism compressive strength shall be taken to not exceed 900 psi for masonry in good condition, 600 psi for masonry in fair condition, and 300 psi for masonry in poor condition.

7.3.2.2 Masonry Elastic Modulus in Compression

Expected values of elastic modulus for masonry in compression, E_{me}, shall be measured using one of the following two methods:

1. Test prisms shall be extracted from an existing wall, transported to a laboratory, and tested in

compression. Stresses and deformations shall be measured to infer modulus values.

2. Two flat jacks shall be inserted into slots cut into mortar bed joints, and pressurized up to nominally one half of the expected masonry compressive strength. Deformations between the two flat jacks shall be measured to infer compressive strain, and thus elastic modulus.

In lieu of prism tests, values for the modulus of elasticity of masonry in compression shall be taken as 550 times the expected masonry compressive strength, f_{me}.

7.3.2.3 Masonry Flexural Tensile Strength

Expected flexural tensile strength, f_{te}, for out-of-plane bending shall be measured using one of the following three methods:

1. Test samples shall be extracted from an existing wall, and subjected to minor-axis bending using the bond-wrench method.

2. Test samples shall be tested in situ using the bond-wrench method.

3. Sample wall panels shall be extracted and subjected to minor-axis bending in accordance with ASTM E 518.

In lieu of material tests, default values of masonry flexural tensile strength for walls or infill panels loaded normal to their plane shall be taken to not exceed 20 psi for masonry in good condition, 10 psi for masonry in fair condition, and zero psi for masonry in poor condition. For masonry constructed after 1960 with cement-based mortars, default values of flexural tensile strength can be based on values from Table 8.3.10.5.1 of BSSC (1995).

Flexural tensile strength for unreinforced masonry (URM) walls subjected to in-plane lateral forces shall be assumed to be equal to that for out-of-plane bending, unless testing is done to define expected tensile strength.

7.3.2.4 Masonry Shear Strength

For URM components, expected masonry shear strength, v_{me}, shall be measured using the in-place shear test. Expected shear strength shall be determined in accordance with Equation 7-1.

$$v_{me} = \frac{0.75\left(0.75 v_{te} + \frac{P_{CE}}{A_n}\right)}{1.5} \tag{7-1}$$

where

P_{CE} = Expected gravity compressive force applied to a wall or pier component stress considering load combinations given in Equations 3-14, 3-15, and 3-16

A_n = Area of net mortared/grouted section, in.^{2}

v_{te} = Average bed-joint shear strength, psi

The 0.75 factor on the v_{te} term may be waived for single wythe masonry, or if the collar joint is known to be absent or in very poor condition.

Values for the mortar shear strength, v_{te}, shall not exceed 100 psi for the determination of v_{me} in Equation 7-1.

Average bed-joint shear strength, v_{te}, shall be determined from individual shear strength test values, v_{to}, in accordance with Equation 7-2.

$$v_{to} = \frac{V_{test}}{A_b} - p_{D+L} \tag{7-2}$$

where V_{test} is the load at first movement of a masonry unit, A_b is the net mortared area of the bed joints above and below the test brick, and p_{D+L} is the estimated gravity stress at the test location.

In lieu of material tests, default values of shear strength of URM components shall be taken to not exceed 27 psi for running bond masonry in good condition, 20 psi for running bond masonry in fair condition, and 13 psi for running bond masonry in poor condition. These values shall also be used for masonry in other than running bond if fully grouted. For masonry in other than running bond and partially grouted or ungrouted, shear strength shall be reduced by 60% of these values. For masonry constructed after 1960 with cement-based mortars, default values of shear strength can be based on values in BSSC (1995) for unreinforced masonry.

The in-place shear test shall not be used to estimate shear strength of reinforced masonry (RM) components. The expected shear strength of RM components shall be in accordance with Section 7.4.4.2A.

7.3.2.5 Masonry Shear Modulus

The expected shear modulus of uncracked, unreinforced, or reinforced masonry, G_{me}, shall be estimated as 0.4 times the elastic modulus in compression. After cracking, the shear modulus shall be taken as a fraction of this value based on the amount of bed joint sliding or the opening of diagonal tension cracks.

7.3.2.6 Strength and Modulus of Reinforcing Steel

The expected yield strength of reinforcing bars, f_{ye}, shall be based on mill test data, or tension tests of actual reinforcing bars taken from the subject building. Tension tests shall be done in accordance with ASTM A 615.

In lieu of tension tests of reinforcing bars, default values of yield stress shall be determined per Section 6.3.2.5. These values shall also be considered as lower bound values, f_y, to be used to estimate lower bound strengths, Q_{CL}.

The expected modulus of elasticity of steel reinforcement, E_{se}, shall be assumed to be 29,000,000 psi.

7.3.2.7 Location and Minimum Number of Tests

The number and location of material tests shall be selected to provide sufficient information to adequately define the existing condition of materials in the building. Test locations shall be identified in those masonry components that are determined to be critical to the primary path of lateral-force resistance.

A visual inspection of masonry condition shall be done in conjunction with any in situ material tests to assess uniformity of construction quality. For masonry with consistent quality, the minimum number of tests for each masonry type, and for each three floors of construction or 3000 square feet of wall surface, shall be three, if original construction records are available that specify material properties, or six, if original construction records are not available. At least two tests should be done per wall, or line of wall elements providing a common resistance to lateral forces. A minimum of eight tests should be done per building.

Tests should be taken at locations representative of the material conditions throughout the entire building, taking into account variations in workmanship at different story levels, variations in weathering of the exterior surfaces, and variations in the condition of the interior surfaces due to deterioration caused by leaks and condensation of water and/or the deleterious effects of other substances contained within the building.

For masonry with perceived inconsistent quality, additional tests shall be done as needed to estimate material strengths in regions where properties are suspected to differ. Nondestructive condition assessment tests per Section 7.3.3.2 may be used to quantify variations in material strengths.

An increased sample size may be adopted to improve the confidence level. The relation between sample size and confidence shall be as defined in ASTM E 22.

If the coefficient of variation in test measurements exceeds 25%, additional tests shall be done. If the variation does not reduce below this limit, use of the test data shall be limited to the Linear Static Procedures of Chapter 3.

If mean values from in situ material tests are less than the default values prescribed in Section 7.3.2, additional tests shall be done. If the mean continues to be less than the default values, the measured values shall be used, and shall be used only with the Linear Static Procedures of Chapter 3.

7.3.3 Condition Assessment

7.3.3.1 Visual Examination

The size and location of all masonry shear and bearing walls shall be determined. The orientation and placement of the walls shall be noted. Overall dimensions of masonry components shall be measured, or determined from plans, including wall heights, lengths, and thicknesses. Locations and sizes of window and door openings shall be measured, or determined from plans. The distribution of gravity loads to bearing walls should be estimated.

The wall type shall be identified as reinforced or unreinforced, composite or noncomposite, and/or grouted, partially grouted, or ungrouted. For RM construction, the size and spacing of horizontal and vertical reinforcement should be estimated. For multiwythe construction, the number of wythes should be noted, as well as the distance between wythes (the thickness of the collar joint or cavity), and the placement of interwythe ties. The condition and attachment of veneer wythes should be noted. For grouted construction, the quality of grout placement should be assessed. For partially grouted walls, the locations of grout placement should be identified.

The type and condition of the mortar and mortar joints shall be determined. Mortar shall be examined for weathering, erosion, and hardness, and to identify the condition of any repointing, including cracks, internal voids, weak components, and/or deteriorated or eroded mortar. Horizontal cracks in bed joints, vertical cracks in head joints and masonry units, and diagonal cracks near openings shall be noted.

The examination shall identify vertical components that are not straight. Bulging or undulations in walls shall be observed, as well as separation of exterior wythes, out-of-plumb walls, and leaning parapets or chimneys.

Connections between masonry walls, and between masonry walls and floors or roofs, shall be examined to identify details and condition. If construction drawings are available, a minimum of three connections shall be inspected for each general connection type (e.g., floor-to-wall, wall-to-wall). If no deviations from the drawings are found, the sample may be considered representative. If drawings are unavailable, or significant deviations are noted between the drawings and constructed work, then a random sample of connections shall be inspected until a representative pattern of connections can be identified.

7.3.3.2 Nondestructive Tests

Nondestructive tests may be used to supplement the visual observations required in Section 7.3.3.1. One, or a combination, of the following nondestructive tests, shall be done to meet the requirements of a comprehensive evaluation as stated in Section 7.3.4:

- ultrasonic pulse velocity
- mechanical pulse velocity
- impact echo
- radiography

The location and number of nondestructive tests shall be in accordance with the requirements of Section 7.3.2.7. Descriptive information concerning these test procedures is provided in the *Commentary,* Section C7.3.3.2.

7.3.3.3 Supplemental Tests

Ancillary tests are recommended, but not required, to enhance the level of confidence in masonry material properties, or to assess condition. These are described in the Commentary to this section.

7.3.4 Knowledge (κ) Factor

In addition to those characteristics specified in Section 2.7.2, a knowledge factor, κ, equal to 0.75, representing a minimum level of knowledge of the structural system, shall be used if a visual examination of masonry structural components is done per the requirements of Section 7.3.3.1. A knowledge factor, κ, equal to 1.00, shall be used only with a comprehensive level of knowledge of the structural system (as defined in Section 2.7.2).

7.4 Engineering Properties of Masonry Walls

This section provides basic engineering information for assessing attributes of structural walls, and includes stiffness assumptions, strength acceptance criteria, and deformation acceptance criteria for the Immediate Occupancy, Life Safety, and Collapse Prevention Performance Levels. Engineering properties given for masonry walls shall be used with the analytical methods prescribed in Chapter 3, unless otherwise noted.

Masonry walls shall be categorized as primary or secondary elements. Walls that are considered to be part of the lateral-force system, and may or may not support gravity loads, shall be primary elements. Walls that are not considered as part of the lateral-force-resisting system, but must remain stable while supporting gravity loads during seismic excitation, shall be secondary elements.

7.4.1 Types of Masonry Walls

The procedures set forth in this section are applicable to building systems comprising any combination of existing masonry walls, masonry walls enhanced for seismic rehabilitation, and new walls added to an existing building for seismic rehabilitation. In addition, any of these three categories of masonry elements can be used in combination with existing, rehabilitated, or new lateral-force-resisting elements of other materials such as steel, concrete, or timber.

When analyzing a system comprising existing masonry walls, rehabilitated masonry walls, and/or new masonry walls, expected values of strength and stiffness shall be used.

7.4.1.1 Existing Masonry Walls

Existing masonry walls considered in Section 7.4 shall include all structural walls of a building system that are in place prior to seismic rehabilitation.

Wall types shall include unreinforced or reinforced; ungrouted, partially grouted, or fully grouted; and composite or noncomposite. Existing walls subjected to lateral forces applied parallel with their plane shall be considered separately from walls subjected to forces applied normal to their plane, as described in Sections 7.4.2 through 7.4.5.

Material properties for existing walls shall be established per Section 7.3.2. Prior to rehabilitation, masonry structural walls shall be assessed for condition per the procedures set forth in Sections 7.3.3.1, 7.3.3.2, or 7.3.3.3. Existing masonry walls shall be assumed to behave in the same manner as new masonry walls, provided that the condition assessment demonstrates equivalent quality of construction.

7.4.1.2 New Masonry Walls

New masonry walls shall include all new elements added to an existing lateral-force-resisting system. Wall types shall include unreinforced or reinforced; ungrouted, partially grouted, or fully grouted; and composite or noncomposite. Design of newly constructed walls shall follow the requirements set forth in BSSC (1995).

When analyzing a system of new and existing walls, expected values of strength and stiffness shall be used for the newly constructed walls. Any capacity reduction factors given in BSSC (1995) shall not be used, and mean values of material strengths shall be used in lieu of lower bound estimates.

New walls subjected to lateral forces applied parallel with their plane shall be considered separately from walls subjected to forces applied normal to their plane, as described in Sections 7.4.2 through 7.4.5.

7.4.1.3 Enhanced Masonry Walls

Enhanced masonry walls shall include existing walls that are rehabilitated with the methods given in this section. Unless stated otherwise, methods are applicable to both unreinforced and reinforced walls, and are intended to improve performance of masonry walls subjected to both in-plane and out-of-plane lateral forces.

Enhanced walls subjected to lateral forces applied parallel with their plane shall be considered separately from walls subjected to forces applied normal to their plane, as described in Sections 7.4.2 through 7.4.5.

A. Infilled Openings

An infilled opening shall be considered to act compositely with the surrounding masonry if the following provisions are met.

1. The sum of the lengths of all openings in the direction of in-plane shear force in a single continuous wall is less than 40% of the overall length of the wall.

2. New and old masonry units shall be interlaced at the boundary of the infilled opening with full toothing, or adequate anchorage shall be provided to give an equivalent shear strength at the interface of new and old units.

Stiffness assumptions, strength criteria, and acceptable deformations for masonry walls with infilled openings shall be the same as given for nonrehabilitated solid masonry walls, provided that differences in elastic moduli and strengths for the new and old masonries are considered for the composite section.

B. Enlarged Openings

Openings in a masonry shear wall may be enlarged by removing portions of masonry above or below windows or doors. This is done to increase the height-to-length aspect ratio of piers so that the limit state may be altered from shear to flexure. This method is only applicable to URM walls.

Stiffness assumptions, strength criteria, and acceptable deformations for URM walls with enlarged openings shall be the same as given for existing perforated masonry walls, provided that the cutting operation does not cause any distress.

C. Shotcrete

An existing masonry wall with an application of shotcrete shall be considered to behave as a composite section, as long as adequate anchorage is provided at the shotcrete-masonry interface for shear transfer. Stresses in the masonry and shotcrete shall be determined considering the difference in elastic moduli for each material. Alternatively, the masonry may be neglected if the new shotcrete layer is designed to resist all of the force, and minor cracking of the masonry is acceptable.

Stiffness assumptions, strength criteria, and acceptable deformations for masonry components with shotcrete shall be the same as for new reinforced concrete components, with due consideration to possible variations in boundary conditions.

D. Coatings for URM Walls

A coated masonry wall shall be considered to behave as a composite section, as long as adequate anchorage is provided at the interface between the coating and the masonry wall. Stresses in the masonry and coating shall be determined considering the difference in elastic moduli for each material. If stresses exceed expected strengths of the coating material, then the coating shall be considered ineffective.

Stiffness assumptions, strength criteria, and acceptable deformations for coated masonry walls shall be the same as for existing URM walls.

E. Reinforced Cores for URM Walls

A reinforced-cored masonry wall shall be considered to behave as a reinforced masonry wall, provided that sufficient bonding exists between the new reinforcement and the grout, and between the grout and the cored surface. Vertical reinforcement shall be anchored at the base of the wall to resist its full tensile strength.

Grout in new reinforced cores should consist of cementitious materials whose hardened properties are compatible with those of the surrounding masonry.

Adequate shear strength must exist, or be provided, so that the strength of the new vertical reinforcement can be developed.

Stiffness assumptions, strength criteria, and acceptable deformations for URM walls with reinforced cores shall be the same as for existing reinforced walls.

F. Prestressed Cores for URM Walls

A prestressed-cored masonry wall with unbonded tendons shall be considered to behave as a URM wall with increased vertical compressive stress.

Losses in prestressing force due to creep and shrinkage of the masonry shall be accounted for.

Stiffness assumptions, strength criteria, and acceptable deformations for URM walls with unbonded prestressing tendons shall be the same as for existing unreinforced masonry walls subjected to vertical compressive stress.

G. Grout Injections

Any grout used for filling voids and cracks shall have strength, modulus, and thermal properties compatible with the existing masonry.

Inspection shall be made during the grouting to ensure that voids are completely filled with grout.

Stiffness assumptions, strength criteria, and acceptable deformations for masonry walls with grout injections shall be the same as for existing unreinforced or reinforced walls.

H. Repointing

Bond strength of new mortar shall be equal to or greater than that of the original mortar. Compressive strength of new mortar shall be equal to or less than that of the original mortar.

Stiffness assumptions, strength criteria, and acceptable deformations for repointed masonry walls shall be the same as for existing masonry walls.

I. Braced Masonry Walls

Masonry walls may be braced with external structural elements to reduce span lengths for out-of-plane bending. Adequate strength shall be provided in the bracing element and connections to resist the transfer of forces from the masonry wall to the bracing element. Out-of-plane deflections of braced walls resulting from the transfer of vertical floor or roof loadings shall be considered.

Stiffness assumptions, strength criteria, and acceptable deformations for braced masonry walls shall be the same as for existing masonry walls. Due consideration shall be given to the reduced span of the masonry wall.

J. Stiffening Elements

Masonry walls may be stiffened with external structural members to increase the out-of-plane stiffness and strength. The stiffening member shall be proportioned to resist a tributary portion of lateral load applied normal to the plane of a masonry wall. Adequate connections at the ends of the stiffening element shall be provided to transfer the reaction of force. Flexibility of the stiffening element shall be considered when estimating lateral drift of a masonry wall panel for Performance Levels.

Stiffness assumptions, strength criteria, and acceptable deformations for stiffened masonry walls shall be the same as for existing masonry walls. Due consideration shall be given to the stiffening action that the new element provides.

7.4.2 URM In-Plane Walls and Piers

Information is given in this section for depicting the engineering properties of URM walls subjected to lateral forces applied parallel with their plane. Requirements of this section shall apply to cantilevered shear walls that are fixed against rotation at their base, and piers between window or door openings that are fixed against rotation at their top and base.

Stiffness and strength criteria are presented that are applicable for use with both the Linear Static and Nonlinear Static Procedures prescribed in Chapter 3.

7.4.2.1 Stiffness

The lateral stiffness of masonry wall and pier components shall be determined based on the minimum net sections of mortared and grouted masonry in accordance with the guidelines of this subsection. The lateral stiffness of masonry walls subjected to lateral in-plane forces shall be determined considering both flexural and shear deformations.

The masonry assemblage of units, mortar, and grout shall be considered to be a homogeneous medium for stiffness computations with an expected elastic modulus in compression, E_{me}, as specified in Section 7.3.2.2.

For linear procedures, the stiffness of a URM wall or pier resisting lateral forces parallel with its plane shall be considered to be linear and proportional with the geometrical properties of the uncracked section.

For nonlinear procedures, the in-plane stiffness of URM walls or piers shall be based on the extent of cracking.

Story shears in perforated shear walls shall be distributed to piers in proportion to the relative lateral uncracked stiffness of each pier.

Stiffnesses for existing, enhanced, and new walls shall be determined using the same principles of mechanics.

7.4.2.2 Strength Acceptance Criteria

Unreinforced masonry walls and piers shall be considered as deformation-controlled components if their expected lateral strength limited by bed-joint sliding shear stress or rocking (the lesser of values given by Equations 7-3 and 7-4) is less than the lower bound lateral strength limited by diagonal tension or toe compressive stress (the lesser of values given by Equations 7-5 or 7-6). Otherwise, these components shall be considered as force-controlled components.

A. Expected Lateral Strength of Walls and Piers

Expected lateral strength of existing URM walls or pier components shall be based on expected bed-joint sliding shear strength, or expected rocking strength, in accordance with Equations 7-3 and 7-4, respectively. The strength of such URM walls or piers shall be the lesser of:

$$Q_{CE} = V_{bjs} = v_{me}A_n \tag{7-3}$$

$$Q_{CE} = V_r = 0.9\alpha P_{CE}\left(\frac{L}{h_{eff}}\right) \tag{7-4}$$

where

A_n = Area of net mortared/grouted section, in.^{2}

h_{eff} = Height to resultant of lateral force

L = Length of wall or pier

P_{CE} = Expected vertical axial compressive force per load combinations in Equations 3-2 and 3-3

v_{me} = Expected bed-joint sliding shear strength per Section 7.3.2.4, psi

V_{bjs} = Lateral strength of wall or pier based on bed-joint shear strength, pounds

V_r = Lateral rocking strength of wall or pier component, pounds

α = Factor equal to 0.5 for fixed-free cantilever wall, or equal to 1.0 for fixed-fixed pier

Expected lateral strength of newly constructed wall or pier components shall be based on the *NEHRP Recommended Provisions* (BSSC, 1995), with the exception that capacity reduction factors shall be taken as equal to 1.0.

B. Lower Bound Lateral Strength of Walls and Piers

Lower bound lateral strength of existing URM walls or pier components shall be limited by diagonal tension stress or toe compressive stress, in accordance with Equations 7-5 and 7-6, respectively. The lateral strength of URM walls or piers shall be the lesser of Q_{CL} values given by these two equations.

If L/h_{eff} is larger than 0.67 and less than 1.00, then:

$$Q_{CL} = V_{dt} = f'_{dt} A_n \left(\frac{L}{h_{eff}}\right) \sqrt{1 + \frac{f_a}{f'_{dt}}} \qquad (7\text{-}5)$$

$$Q_{CL} = V_{tc} = \alpha P_{CL} \left(\frac{L}{h_{eff}}\right) \left(1 - \frac{f_a}{0.7 f'_m}\right) \qquad (7\text{-}6)$$

where A_n, h_{eff}, L, and α are the same as given for Equations 7-3 and 7-4, and:

f_a = Upper bound of vertical axial compressive stress from Equation 3-2, psi

f'_{dt} = Lower bound of masonry diagonal tension strength, psi

f'_m = Lower bound of masonry compressive strength, psi

P_{CL} = Lower bound of vertical compressive force from load combination of Equation 3-3, pounds

V_{dt} = Lateral strength limited by diagonal tension stress, pounds

V_{tc} = Lateral strength limited by toe compressive stress, pounds

For determination of V_{dt}, the bed-joint shear strength, v_{me}, may be substituted for the diagonal tension strength, f'_{dt}, in Equation 7-5.

The lower bound masonry compressive strength, f'_m, shall be taken as the expected strength, f_{me}, determined per Section 7.3.2.1, divided by 1.6.

If the Linear Static Procedures of Section 3.3 are used, lateral forces from gravity and seismic effects shall be less than the lower bound lateral strength, Q_{CL}, as required by Equation 3-19.

C. Lower Bound Vertical Compressive Strength of Walls and Piers

Lower bound vertical compressive strength of existing URM walls or pier components shall be limited by masonry compressive stress per Equation 7-7.

$$Q_{CL} = P_c = 0.80(0.85 f'_m A_n) \qquad (7\text{-}7)$$

where f'_m is equal to the expected strength, f_{me}, determined per Section 7.3.2.1, divided by 1.6.

If the Linear Static Procedures of Section 3.3 are used, vertical forces from gravity and seismic effects shall be less than the lower bound lateral strength, Q_{CL}, as stated in Equation 3-19.

7.4.2.3 Deformation Acceptance Criteria

A. Linear Procedures

Table 7-1 Linear Static Procedure—m Factors for URM In-Plane Walls and Piers

Limiting Behavioral Mode	*m* Factors				
	Primary			Secondary	
	IO	LS	CP	LS	CP
Bed-Joint Sliding	1	3	4	6	8
Rocking	$(1.5h_{eff}/L)>1$	$(3h_{eff}/L)>1.5$	$(4h_{eff}/L)>2$	$(6h_{eff}/L)>3$	$(8h_{eff}/L)>4$

Note: Interpolation is permitted between table values.

If the linear procedures of Section 3.3 are used, the product of expected strength, Q_{CE}, of those components classified as deformation-controlled, multiplied by *m* factors given in Table 7-1 for particular Performance Levels and κ factors given in Section 2.7.2, shall exceed the sum of unreduced seismic forces, Q_E, and gravity forces, Q_G, per Equation 3-18. The limiting behavior mode in Table 7-1 shall be identified from the lower of the two expected strengths as determined from Equations 7-3 and 7-4.

For determination of *m* factors from Table 7-1, the vertical compressive stress, f_{ae}, shall be based on an expected value of gravity compressive force given by the load combinations given in Equations 3-2 and 3-3.

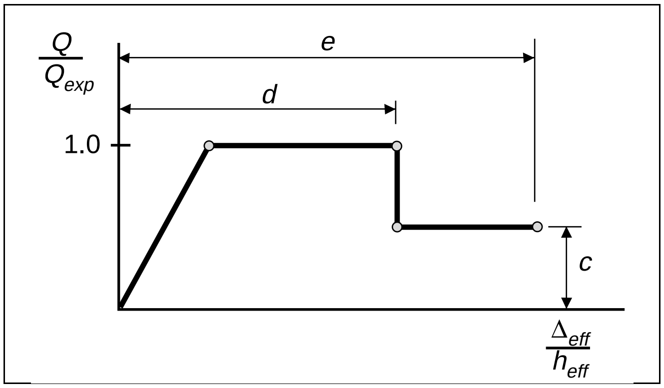

Figure 7-1 Idealized Force-Deflection Relation for Walls, Pier Components, and Infill Panels

B. Nonlinear Procedures

If the Nonlinear Static Procedure given in Section 3.3.3 is used, deformation-controlled wall and pier components shall be assumed to deflect to nonlinear lateral drifts as given in Table 7-2. Variables *d* and *e*, representing nonlinear deformation capacities for primary and secondary components, are expressed in terms of story drift ratio percentages, as defined in Figure 7-1.The limiting behavior mode in Table 7-2 shall be identified from the lower of the two expected strengths as determined from Equations 7-3 and 7-4.

For components of primary lateral-force-resisting elements, collapse shall be considered at lateral drift percentages exceeding values of *d* in the table, and the Life Safety Performance Level shall be considered at approximately 75% of the *d* value. For components of secondary elements, collapse shall be considered at lateral drift percentages exceeding the values of *e* in the table, and the Life Safety Performance Level shall be considered at approximately 75% of the *e* value in the table. Drift percentages based on these criteria are given in Table 7-2.

If the Nonlinear Dynamic Procedure given in Section 3.3.4 is used, nonlinear force-deflection relations for wall and pier components shall be established based on the information given in Table 7-2, or on a more comprehensive evaluation of the hysteretic characteristics of those components.

7.4.3 URM Out-of-Plane Walls

As required by Section 2.11.7, URM walls shall be considered to resist out-of-plane excitation as isolated

Table 7-2 Nonlinear Static Procedure—Simplified Force-Deflection Relations for URM In-Plane Walls and Piers

				Acceptance Criteria				
Limiting Behavioral Mode				**Primary**			**Secondary**	
	c **%**	***d*** **%**	***e*** **%**	**IO %**	**LS %**	**CP %**	**LS %**	**CP %**
Bed-Joint Sliding	0.6	0.4	0.8	0.1	0.3	0.4	0.6	0.8
Rocking	0.6	$0.4h_{eff}/L$	$0.8h_{eff}/L$	0.1	$0.3h_{eff}/L$	$0.4h_{eff}/L$	$0.6h_{eff}/L$	$0.8h_{eff}/L$

Note: Interpolation is permitted between table values.

components spanning between floor levels, and/or spanning horizontally between columns or pilasters. Out-of-plane walls shall not be analyzed with the Linear or Nonlinear Static Procedures prescribed in Chapter 3.

7.4.3.1 Stiffness

The stiffness of out-of-plane walls shall be neglected with analytical models of the global structural system if in-plane walls or infill panels exist, or are placed, in the orthogonal direction.

7.4.3.2 Strength Acceptance Criteria

The Immediate Occupancy Performance Level shall be limited by flexural cracking of out-of-plane walls. Unless arching action is considered, flexural cracking shall be limited by the expected tensile stress values given in Section 7.3.2.3 for existing walls and in BSSC (1995) for new construction.

Arching action shall be considered if, and only if, surrounding floor, roof, column, or pilaster elements have sufficient stiffness and strength to resist thrusts from arching of a wall panel, and a condition assessment has been done to ensure that there are no gaps between a wall panel and the adjacent structure.

Due consideration shall be given to the condition of the collar joint when estimating the effective thickness of a wall.

7.4.3.3 Deformation Acceptance Criteria

The Life Safety and Collapse Prevention Levels permit flexural cracking in URM walls subjected to out-of-plane loading, provided that cracked wall segments will remain stable during dynamic excitation. Stability shall be checked using analytical time-step integration models with realistic depiction of acceleration time histories at the top and base of a wall panel. Walls spanning vertically, with a height-to-thickness (h/t) ratio less than that given in Table 7-3, need not be checked for dynamic stability.

Table 7-3 Permissible h/t Ratios for URM Out-of-Plane Walls

Wall Types	$S_{X1} \leq 0.24g$	$0.24g < S_{X1} \leq 0.37g$	$0.37g < S_{X1} \leq 0.5g$
Walls of one-story buildings	20	16	13
First-story wall of multistory building	20	18	15
Walls in top story of multistory building	14	14	9
All other walls	20	16	13

7.4.4 Reinforced Masonry In-Plane Walls and Piers

7.4.4.1 Stiffness

The stiffness of a reinforced in-plane wall or pier component shall be based on:

- The uncracked section, when an analysis is done to show that the component will not crack when subjected to expected levels of axial and lateral force
- The cracked section, when an analysis is done to show that the component will crack when subjected to expected levels of axial and lateral force

Stiffnesses for existing and new walls shall be assumed to be the same.

7.4.4.2 Strength Acceptance Criteria for Reinforced Masonry (RM)

Strength of RM wall or pier components in flexure, shear, and axial compression shall be determined per the requirements of this section. The assumptions, procedures, and requirements of this section shall apply to both existing and newly constructed RM wall or pier components.

Reinforced masonry walls and piers shall be considered as deformation-controlled components if their expected lateral strength for flexure per Section 7.4.4.2A is less than the lower bound lateral strength limited by shear per Section 7.4.4.2B. Vertical compressive behavior of reinforced masonry wall or pier components shall be assumed to be a force-controlled action. Methods for determining lower bound axial compressive strength are given in Section 7.4.4.2D.

A. Expected Flexural Strength of Walls and Piers

Expected flexural strength of an RM wall or pier shall be determined on the basis of the following assumptions.

- Stress in reinforcement below the expected yield strength, f_{ye}, shall be taken as the modulus of elasticity, E_{se}, times the steel strain. For reinforcement strains larger than those corresponding to the expected yield strength, the stress in the reinforcement shall be considered independent of strain and equal to the expected yield strength, f_{ye}.

- Tensile strength of masonry shall be neglected in calculating the flexural strength of a reinforced masonry cross section.

- Flexural compression stress in masonry shall be assumed to be distributed across an equivalent rectangular stress block. Masonry stress of 0.85 times the expected compressive strength, f_{me}, shall be distributed uniformly over an equivalent compression zone bounded by edges of the cross section and with a depth equal to 85% of the depth from the neutral axis to the fiber of maximum compressive strain.

- Strains in the reinforcement and masonry shall be considered linear across the wall or pier cross section. For purposes of determining forces in reinforcing bars distributed across the section, the maximum compressive strain in the masonry shall be assumed to be equal to 0.003.

B. Lower-Bound Shear Strength of Walls and Piers

Lower-bound shear strength of RM wall or pier components, V_{CL}, shall be determined using Equation 7-8.

$$Q_{CL} = V_{CL} = V_{mL} + V_{sL} \tag{7-8}$$

where:

V_{mL} = Lower bound shear strength provided by masonry, lb

V_{sL} = Lower bound shear strength provided by reinforcement, lb

The lower bound shear strength of an RM wall or pier shall not exceed shear forces given by Equations 7-9 and 7-10.

For M/Vd_v less than 0.25:

$$V_{CL} \le 6\sqrt{f'_m}\,A_n \tag{7-9}$$

For M/Vd_v greater than or equal to 1.00:

$$V_{CL} \le 4\sqrt{f'_m}\,A_n \tag{7-10}$$

where:

A_n = Area of net mortared/grouted section, in.^{2}

f'_m = Compressive strength of masonry, psi

M = Moment on the masonry section, in.-lb

V = Shear on the masonry section, lb

d_v = Wall length in direction of shear force, in.

Lower-bound shear strength, V_{mL}, resisted by the masonry shall be determined using Equation 7-11.

$$V_{mL} = \left[4.0 - 1.75\left(\frac{M}{Vd_v}\right)\right]A_n\sqrt{f_m} + 0.25P_{CL} \quad (7\text{-}11)$$

where M/Vd_v need not be taken greater than 1.0, and P_{CL} is the lower-bound vertical compressive force in pounds based on the load combinations given in Equations 3-2 and 3-3.

Lower-bound shear strength, V_{sL}, resisted by the reinforcement shall be determined using Equation 7-12.

$$V_{sL} = 0.5\left(\frac{A_v}{s}\right)f_y d_v \quad (7\text{-}12)$$

where:

A_v = Area of shear reinforcement, in.^{2}

s = Spacing of shear reinforcement, in.

f_y = Lower-bound yield strength of shear reinforcement, psi

C. Strength Considerations for Flanged Walls

Wall intersections shall be considered effective in transferring shear when either condition (1) or (2), and condition (3), as noted below, are met:

1. The face shells of hollow masonry units are removed and the intersection is fully grouted.

2. Solid units are laid in running bond, and 50% of the masonry units at the intersection are interlocked.

3. Reinforcement from one intersecting wall continues past the intersection a distance not less than 40 bar diameters or 24 inches.

The width of flange considered effective in compression on each side of the web shall be taken as equal to six times the thickness of the web, or shall be equal to the actual flange on either side of the web wall, whichever is less.

The width of flange considered effective in tension on each side of the web shall be taken as equal to 3/4 of the wall height, or shall be equal to the actual flange on either side of the web wall, whichever is less.

D. Lower Bound Vertical Compressive Strength of Walls and Piers

Lower bound vertical compressive strength of existing RM walls or pier components shall be determined using Equation 7-13.

$$Q_{CL} = P_c = 0.8[0.85f'_m(A_n - A_s) + A_s f_y] \quad (7\text{-}13)$$

where:

f'_m = Lower bound masonry compressive strength equal to expected strength, f_{me}, determined per Section 7.3.2.1, divided by 1.6

f_y = Lower bound reinforcement yield strength per Section 7.3.2.6

If the Linear Static Procedures of Section 3.3.1 are used, vertical forces from gravity and seismic effects shall be less than the lower bound lateral strength, Q_{CL}, as required by Equation 3-19.

7.4.4.3 Deformation Acceptance Criteria

A. Linear Procedures

If the linear procedures of Section 3.3 are used, the product of expected strength, Q_{CE}, of those components classified as deformation-controlled, multiplied by *m* factors given in Table 7-4 for particular Performance Levels and κ factors given in Section 2.7.2, shall exceed the sum, Q_{UD},of unreduced seismic forces, Q_E, and gravity forces, Q_G, as in Equations 3-14 and 3-18.

For determination of *m* factors from Table 7-4, the ratio of vertical compressive stress to expected compressive strength, f_{ae}/f_{me}, shall be based on an expected value of gravity compressive force per the load combinations given in Equations 3-2 and 3-3.

B. Nonlinear Procedures

If the Nonlinear Static Procedure given in Section 3.3.3 is used, deformation-controlled wall and pier components shall be assumed to deflect to nonlinear lateral drifts as given in Table 7-5. Variables *d* and *e*, representing nonlinear deformation capacities for primary and secondary components, are expressed in terms of story drift ratio percentages as defined in Figure 7-1.

For determination of the *c*, *d*, and *e* values and the acceptable drift levels using Table 7-5, the vertical compressive stress, f_{ae}, shall be based on an expected value of gravity compressive force per the load combinations given in Equations 3-2 and 3-3.

For components of primary lateral-force-resisting elements, collapse shall be considered at lateral drift percentages exceeding values of *d* in Table 7-5, and the Life Safety Performance Level shall be considered at approximately 75% of the *d* value. For components of secondary elements, collapse shall be considered at lateral drift percentages exceeding the values of *e* in the table, and the Life Safety Performance Level shall be considered at approximately 75% of the *e* value in the table. Story drift ratio percentages based on these criteria are given in Table 7-5.

If the Nonlinear Dynamic Procedure given in Section 3.3.4 is used, nonlinear force-deflection relations for wall and pier components shall be established based on the information given in Table 7-5, or on a more comprehensive evaluation of the hysteretic characteristics of those components.

Acceptable deformations for existing and new walls shall be assumed to be the same.

7.4.5 RM Out-of-Plane Walls

As required by Section 2.11.7, RM walls shall be considered to resist out-of-plane excitation as isolated components spanning between floor levels, and/or spanning horizontally between columns or pilasters. Out-of-plane walls shall not be analyzed with the Linear or Nonlinear Static Procedures prescribed in Chapter 3, but shall resist lateral inertial forces as given in Section 2.11.7, or respond to earthquake motions as determined with the Nonlinear Dynamic Procedure and satisfy the deflection criteria given in Section 7.4.5.3.

7.4.5.1 Stiffness

Out-of-plane RM walls shall be considered as local elements spanning between individual story levels.

The stiffness of out-of-plane walls shall be neglected with analytical models of the global structural system if in-plane walls exist or are placed in the orthogonal direction.

Uncracked sections based on the net mortared/grouted area shall be considered for determination of geometrical properties, provided that net flexural tensile stress does not exceed expected tensile strength, f_{te}, per Section 7.3.2.3. Stiffness shall be based on a cracked section for a wall whose net flexural tensile stress exceeds expected tensile strength.

Stiffnesses for existing and new reinforced out-of-plane walls shall be assumed to be the same.

7.4.5.2 Strength Acceptance Criteria

Out-of-plane RM walls shall be sufficiently strong in flexure to resist the transverse loadings prescribed in Section 2.11.7 for all Performance Levels. Expected flexural strength shall be based on the assumptions given in Section 7.4.4.2A. For walls with an h/t ratio exceeding 20, the effects of deflections on moments shall be considered.

Strength of new walls and existing walls shall be assumed to be the same.

7.4.5.3 Deformation Acceptance Criteria

If the Nonlinear Dynamic Procedure is used, the following performance criteria shall be based on the maximum deflection normal to the plane of a transverse wall.

- The Immediate Occupancy Performance Level shall be met when significant visual cracking of an RM wall occurs. This limit state shall be assumed to occur at a lateral story drift ratio of approximately 2%.
- The Life Safety Performance Level shall be met when masonry units are dislodged and fall out of the wall. This limit state shall be assumed to occur at a lateral drift of a story panel equal to approximately 3%.
- The Collapse Prevention Performance Level shall be met when the post-earthquake damage state is on the verge of collapse. This limit state shall be assumed to occur at a lateral story drift ratio of approximately 5%.

Acceptable deformations for existing and new walls shall be assumed to be the same.

7.5 Engineering Properties of Masonry Infills

This section provides basic engineering information for assessing attributes of masonry infill panels, including

Table 7-4 Linear Static Procedure—m Factors for Reinforced Masonry In-Plane Walls

			m Factors				
			Primary			Secondary	
f_{ae}/f_{me}	L/h_{eff}	$\rho_g f_{ye}/f_{me}$	IO	LS	CP	LS	CP
0.00	0.5	0.01	4.0	7.0	8.0	8.0	10.0
		0.05	2.5	5.0	6.5	8.0	10.0
		0.20	1.5	2.0	2.5	4.0	5.0
	1.0	0.01	4.0	7.0	8.0	8.0	10.0
		0.05	3.5	6.5	7.5	8.0	10.0
		0.20	1.5	3.0	4.0	6.0	8.0
	2.0	0.01	4.0	7.0	8.0	8.0	10.0
		0.05	3.5	6.5	7.5	8.0	10.0
		0.20	2.0	3.5	4.5	7.0	9.0
0.038	0.5	0.01	3.0	6.0	7.5	8.0	10.0
		0.05	2.0	3.5	4.5	7.0	9.0
		0.20	1.5	2.0	2.5	4.0	5.0
	1.0	0.01	4.0	7.0	8.0	8.0	10.0
		0.05	2.5	5.0	6.5	8.0	10.0
		0.20	1.5	2.5	3.5	5.0	7.0
	2.0	0.01	4.0	7.0	8.0	8.0	10.0
		0.05	3.5	6.5	7.5	8.0	10.0
		0.20	1.5	3.0	4.0	6.0	8.0
0.075	0.5	0.01	2.0	3.5	4.5	7.0	9.0
		0.05	1.5	3.0	4.0	6.0	8.0
		0.20	1.0	2.0	2.5	4.0	5.0
	1.0	0.01	2.5	5.0	6.5	8.0	10.0
		0.05	2.0	3.5	4.5	7.0	9.0
		0.20	1.5	2.5	3.5	5.0	7.0
	2.0	0.01	4.0	7.0	8.0	8.0	10.0
		0.05	2.5	5.0	6.5	8.0	10.0
		0.20	1.5	3.0	4.0	4.0	8.0

Note: Interpolation is permitted between table values.

stiffness assumptions, and the strength acceptance and deformation acceptance criteria for the Immediate Occupancy, Life Safety, and Collapse Prevention Performance Levels. Engineering properties given for masonry infills shall be used with the analytical methods prescribed in Chapter 3, unless noted otherwise.

Masonry infill panels shall be considered as primary elements of a lateral-force-resisting system. If the surrounding frame can remain stable following the loss of an infill panel, infill panels shall not be subject to limits set by the Collapse Prevention Performance Level.

Table 7-5 Nonlinear Static Procedure—Simplified Force-Deflection Relations for Reinforced Masonry Shear Walls

						Acceptance Criteria				
						Primary			Secondary	
f_{ae}/f_{me}	L/h_{eff}	$\rho_g f_{ye}/f_{me}$	c	d %	e %	IO %	LS %	CP %	LS %	CP %
0.00	0.5	0.01	0.5	2.6	5.3	1.0	2.0	2.6	3.9	5.3
		0.05	0.6	1.1	2.2	0.4	0.8	1.1	1.6	2.2
		0.20	0.7	0.5	1.0	0.2	0.4	0.5	0.7	1.0
	1.0	0.01	0.5	2.1	4.1	0.8	1.6	2.1	3.1	4.1
		0.05	0.6	0.8	1.6	0.3	0.6	0.8	1.2	1.6
		0.20	0.7	0.3	0.6	0.1	0.2	0.3	0.5	0.6
	2.0	0.01	0.5	1.6	3.3	0.6	1.2	1.6	2.5	3.3
		0.05	0.6	0.6	1.3	0.2	0.5	0.6	0.9	1.3
		0.20	0.7	0.2	0.4	0.1	0.2	0.2	0.3	0.4
0.038	0.5	0.01	0.4	1.0	2.0	0.4	0.8	1.0	1.5	2.0
		0.05	0.5	0.7	1.4	0.3	0.5	0.7	1.0	1.4
		0.20	0.6	0.4	0.9	0.2	0.3	0.4	0.7	0.9
	1.0	0.01	0.4	0.8	1.5	0.3	0.6	0.8	1.1	1.5
		0.05	0.5	0.5	1.0	0.2	0.4	0.5	0.7	1.0
		0.20	0.6	0.3	0.6	0.1	0.2	0.3	0.4	0.6
	2.0	0.01	0.4	0.6	1.2	0.2	0.4	0.6	0.9	1.2
		0.05	0.5	0.4	0.7	0.1	0.3	0.4	0.5	0.7
		0.20	0.6	0.2	0.4	0.1	0.1	0.2	0.3	0.4
0.075	0.5	0.01	0.3	0.6	1.2	0.2	0.5	0.6	0.9	1.2
		0.05	0.4	0.5	1.0	0.2	0.4	0.5	0.8	1.0
		0.20	0.5	0.4	0.8	0.1	0.3	0.4	0.6	0.8
	1.0	0.01	0.3	0.4	0.9	0.2	0.3	0.4	0.7	0.9
		0.05	0.4	0.4	0.7	0.1	0.3	0.4	0.5	0.7
		0.20	0.5	0.2	0.5	0.1	0.2	0.2	0.4	0.5
	2.0	0.01	0.3	0.3	0.7	0.1	0.2	0.3	0.5	0.7
		0.05	0.4	0.3	0.5	0.1	0.2	0.3	0.4	0.5
		0.20	0.5	0.2	0.3	0.1	0.1	0.2	0.2	0.3

Note: Interpolation is permitted between table values.

7.5.1 Types of Masonry Infills

Procedures set forth in this section are applicable to existing panels, panels enhanced for seismic rehabilitation, and new panels added to an existing frame. Infills shall include panels built partially or fully within the plane of steel or concrete frames, and bounded by beams and columns around their perimeters.

Infill panel types considered in these *Guidelines* include unreinforced clay-unit masonry, concrete masonry, and hollow-clay tile masonry. Infills made of stone or glass block are not addressed.

Infill panels that are considered to be isolated from the surrounding frame must have sufficient gaps at top and sides to accommodate maximum lateral frame deflections. Isolated panels shall be restrained in the transverse direction to insure stability under normal forces. Panels that are in tight contact with the frame elements on all four sides are termed shear infill panels. For panels to be considered under this designation, any gaps between an infill and a surrounding frame shall be filled to provide tight contact.

Frame members and connections surrounding infill panels shall be evaluated for frame-infill interaction effects. These effects shall include forces transferred from an infill panel to beams, columns, and connections, and bracing of frame members across a partial length.

7.5.1.1 Existing Masonry Infills

Existing masonry infills considered in this section shall include all structural infills of a building system that are in place prior to seismic rehabilitation.

Infill types included in this section consist of unreinforced and ungrouted panels, and composite or noncomposite panels. Existing infill panels subjected to lateral forces applied parallel with their plane shall be considered separately from infills subjected to forces applied normal to their plane, as described in Sections 7.5.2 and 7.5.3.

Material properties for existing infills shall be established per Section 7.3.2. Prior to rehabilitation, masonry infills shall be assessed for condition per procedures set forth in Sections 7.3.3.1, 7.3.3.2, or 7.3.3.3. Existing masonry infills shall be assumed to behave the same as new masonry infills, provided that a condition assessment demonstrates equivalent quality of construction.

7.5.1.2 New Masonry Infills

New masonry infills shall include all new panels added to an existing lateral-force-resisting system for structural rehabilitation. Infill types shall include unreinforced, ungrouted, reinforced, grouted and partially grouted, and composite or noncomposite.

When analyzing a system of new infills, expected values of strength and stiffness shall be used. No capacity reduction factors shall be used, and expected values of material strengths shall be used in lieu of lower bound estimates.

7.5.1.3 Enhanced Masonry Infills

Enhanced masonry infill panels shall include existing infills that are rehabilitated with the methods given in this section. Unless stated otherwise, methods are applicable to unreinforced infills, and are intended to improve performance of masonry infills subjected to both in-plane and out-of-plane lateral forces.

Masonry infills that are enhanced in accordance with the minimum standards of this section shall be considered using the same Analysis Procedures and performance criteria as for new infills.

Guidelines from the following sections, pertaining to enhancement methods for unreinforced masonry walls, shall also apply to unreinforced masonry infill panels: (1) “Infilled Openings,” Section 7.4.1.3A; (2) “Shotcrete,” Section 7.4.1.3C; (3) “Coatings for URM Walls,” Section 7.4.1.3D; (4) “Grout Injections,” Section 7.4.1.3G; (5) “Repointing,” Section 7.4.1.3H; and (6) “Stiffening Elements,” Section 7.4.1.3J. In addition, the following two enhancement methods shall also apply to masonry infill panels.

A. Boundary Restraints for Infill Panels

Infill panels not in tight contact with perimeter frame members shall be restrained for out-of-plane forces. This may be accomplished by installing steel angles or plates on each side of the infills, and welding or bolting the angles or plates to the perimeter frame members.

B. Joints Around Infill Panels

Gaps between an infill panel and the surrounding frame shall be filled if integral infill-frame action is assumed for in-plane response.

7.5.2 In-Plane Masonry Infills

7.5.2.1 Stiffness

The elastic in-plane stiffness of a solid unreinforced masonry infill panel prior to cracking shall be represented with an equivalent diagonal compression strut of width, a, given by Equation 7-14. The equivalent strut shall have the same thickness and modulus of elasticity as the infill panel it represents.

$$a = 0.175(\lambda_1 h_{col})^{-0.4} r_{inf} \qquad (7\text{-}14)$$

where

$$\lambda_1 = \left[\frac{E_{me} t_{inf} \sin 2\theta}{4 E_{fe} I_{col} h_{inf}}\right]^{\frac{1}{4}}$$

and

h_{col} = Column height between centerlines of beams, in.

h_{inf} = Height of infill panel, in.

E_{fe} = Expected modulus of elasticity of frame material, psi

E_{me} = Expected modulus of elasticity of infill material, psi

I_{col} = Moment of inertia of column, in.^4

L_{inf} = Length of infill panel, in.

r_{inf} = Diagonal length of infill panel, in.

t_{inf} = Thickness of infill panel and equivalent strut, in.

θ = Angle whose tangent is the infill height-to-length aspect ratio, radians

λ_1 = Coefficient used to determine equivalent width of infill strut

For noncomposite infill panels, only the wythes in full contact with the frame elements shall be considered when computing in-plane stiffness, unless positive anchorage capable of transmitting in-plane forces from frame members to all masonry wythes is provided on all sides of the walls.

Stiffness of cracked unreinforced masonry infill panels shall be represented with equivalent struts, provided that the strut properties are determined from detailed analyses that consider the nonlinear behavior of the infilled frame system after the masonry is cracked.

The equivalent compression strut analogy shall be used to represent the elastic stiffness of a perforated unreinforced masonry infill panel, provided that the equivalent strut properties are determined from appropriate stress analyses of infill walls with representative opening patterns.

Stiffnesses for existing and new infills shall be assumed to be the same.

7.5.2.2 Strength Acceptance Criteria

A. Infill Shear Strength

The transfer of story shear across a masonry infill panel confined within a concrete or steel frame shall be considered as a deformation-controlled action. Expected in-plane panel shear strength shall be determined per the requirements of this section.

Expected infill shear strength, V_{ine}, shall be calculated as the product of the net mortared and grouted area of the infill panel, A_{ni}, times the expected shear strength of the masonry, f_{vie}, in accordance with Equation 7-15.

$$Q_{CE} = V_{ine} = A_{ni} f_{vie} \qquad (7\text{-}15)$$

where:

A_{ni} = Area of net mortared/grouted section across infill panel, in.^2

f_{vie} = Expected shear strength of masonry infill, psi

Expected shear strength of existing infills, f_{vie}, shall be taken to not exceed the expected masonry bed-joint shear strength, v_{me}, as determined per Section 7.3.2.4.

Shear strength of newly constructed infill panels, f_{vie}, shall not exceed values given in Section 8.7.4 of BSSC (1995) for zero vertical compressive stress.

For noncomposite infill panels, only the wythes in full contact with the frame elements shall be considered when computing in-plane strength, unless positive anchorage capable of transmitting in-plane forces from frame members to all masonry wythes is provided on all sides of the walls.

B. Required Strength of Column Members Adjacent to Infill Panels

Unless a more rigorous analysis is done, the expected flexural and shear strengths of column members adjacent to an infill panel shall exceed forces resulting from one of the following conditions:

1. The application of the horizontal component of the expected infill strut force applied at a distance, l_{ceff}, from the top or bottom of the infill panel equal to:

$$l_{ceff} = \frac{a}{\cos\theta_c} \tag{7-16}$$

where:

$$\tan\theta_c = \frac{h_{inf} - \frac{a}{\cos\theta_c}}{L_{inf}} \tag{7-17}$$

2. The shear force resulting from development of expected column flexural strengths at the top and bottom of a column with a reduced height equal to l_{ceff}

The reduced column length, l_{ceff}, in Equation 7-16 shall be equal to the clear height of opening for a captive column braced laterally with a partial height infill.

The requirements of this section shall be waived if the expected masonry shear strength, v_{me}, as measured per test procedures of Section 7.3.2.4, is less than 50 psi.

C. Required Strength of Beam Members Adjacent to Infill Panels

The expected flexural and shear strengths of beam members adjacent to an infill panel shall exceed forces resulting from one of the following conditions:

1. The application of the vertical component of the expected infill strut force applied at a distance, l_{beff}, from the top or bottom of the infill panel equal to:

$$l_{ceff} = \frac{a}{\sin\theta_b} \tag{7-18}$$

where:

$$\tan\theta_b = \frac{h_{inf}}{L_{inf} - \frac{a}{\sin\theta_b}} \tag{7-19}$$

2. The shear force resulting from development of expected beam flexural strengths at the ends of a beam member with a reduced length equal to l_{beff}

The requirements of this section shall be waived if the expected masonry shear strength, v_{me}, as measured per test procedures of Section 7.3.2.4, is less than 50 psi.

7.5.2.3 Deformation Acceptance Criteria

A. Linear Procedures

If the linear procedures of Section 3.3.1 are used, the product of expected infill strength, V_{ine}, multiplied by m factors given in Table 7-6 for particular Performance Levels and κ factors given in Section 2.7.2, shall exceed the sum of unreduced seismic forces, Q_E, and gravity forces, Q_G, per Equation 3-18. For the case of an infill panel, Q_E shall be the horizontal component of the unreduced axial force in the equivalent strut member.

For determination of m factors per Table 7-6, the ratio of frame to infill strengths shall be determined considering the expected lateral strength of each component. If the expected frame strength is less than 0.3 times the expected infill strength, the confining effects of the frame shall be neglected and the masonry component shall be evaluated as an individual wall component per Sections 7.4.2 or 7.4.4.

B. Nonlinear Procedures

If the Nonlinear Static Procedure given in Section 3.3.3 is used, infill panels shall be assumed to deflect to nonlinear lateral drifts as given in Table 7-7. The variable d, representing nonlinear deformation capacities, is expressed in terms of story drift ratio in percent as defined in Figure 7-1.

For determination of the d values and the acceptable drift levels using Table 7-7, the ratio of frame to infill strengths shall be determined considering the expected lateral strength of each component. If the expected frame strength is less than 0.3 times the expected infill strength, the confining effects of the frame shall be neglected and the masonry component shall be

Table 7-6 Linear Static Procedure—m Factors for Masonry Infill Panels

$\beta = \frac{V_{fre}}{V_{ine}}$	$\frac{L_{inf}}{h_{inf}}$	***m* Factors**		
		IO	**LS**	**CP**
$0.3 \le \beta < 0.7$	0.5	1.0	4.0	n.a.
	1.0	1.0	3.5	n.a.
	2.0	1.0	3.0	n.a.
$0.7 \le \beta < 1.3$	0.5	1.5	6.0	n.a.
	1.0	1.2	5.2	n.a.
	2.0	1.0	4.5	n.a.
$\beta \ge 1.3$	0.5	1.5	8.0	n.a.
	1.0	1.2	7.0	n.a.
	2.0	1.0	6.0	n.a.

Note: Interpolation is permitted between table values.

evaluated as an individual wall component per Sections 7.4.2 or 7.4.4.

The Immediate Occupancy Performance Level shall be met when significant visual cracking of an unreinforced masonry infill occurs. The Life Safety Performance Level shall be met when substantial cracking of the masonry infill occurs, and the potential for the panel, or some portion of it, to drop out of the frame is high. Acceptable story drift ratio percentages corresponding to these general Performance Levels are given in Table 7-7.

If the surrounding frame can remain stable following the loss of an infill panel, infill panels shall not be subject to limits set by the Collapse Prevention Performance Level.

If the Nonlinear Dynamic Procedure given in Section 3.3.4 is used, nonlinear force-deflection relations for infill panels shall be established based on the information given in Table 7-7, or on a more comprehensive evaluation of the hysteretic characteristics of those components.

Acceptable deformations for existing and new infills shall be assumed to be the same.

Table 7-7 Nonlinear Static Procedure—Simplified Force-Deflection Relations for Masonry Infill Panels

$\beta = \frac{V_{fre}}{V_{ine}}$	$\frac{L_{inf}}{h_{inf}}$	**c**	**d %**	**e %**	**Acceptance Criteria**	
					LS %	**CP %**
$0.3 \le \beta < 0.7$	0.5	n.a.	0.5	n.a.	0.4	n.a.
	1.0	n.a.	0.4	n.a.	0.3	n.a.
	2.0	n.a.	0.3	n.a.	0.2	n.a.
$0.7 \le \beta < 1.3$	0.5	n.a.	1.0	n.a.	0.8	n.a.
	1.0	n.a.	0.8	n.a.	0.6	n.a.
	2.0	n.a.	0.6	n.a.	0.4	n.a.
$\beta \ge 1.3$	0.5	n.a.	1.5	n.a.	1.1	n.a.
	1.0	n.a.	1.2	n.a.	0.9	n.a.
	2.0	n.a.	0.9	n.a.	0.7	n.a.

Note: Interpolation is permitted between table values.

7.5.3 Out-of-Plane Masonry Infills

Unreinforced infill panels with h_{inf}/t_{inf} ratios less than those given in Table 7-8, and meeting the requirements for arching action given in the following section, need not be analyzed for transverse seismic forces.

7.5.3.1 Stiffness

Out-of-plane infill panels shall be considered as local elements spanning vertically between floor levels and/ or horizontally across bays of frames.

Table 7-8 ***Maximum h_{inf}/t_{inf} Ratios for which No Out-of-Plane Analysis is Necessary***

	Low Seismic Zone	Moderate Seismic Zone	High Seismic Zone
IO	14	13	8
LS	15	14	9
CP	16	15	10

The stiffness of infill panels subjected to out-of-plane forces shall be neglected with analytical models of the global structural system if in-plane walls or infill panels exist, or are placed, in the orthogonal direction.

Flexural stiffness for uncracked masonry infills subjected to transverse forces shall be based on the minimum net sections of mortared and grouted masonry. Flexural stiffness for unreinforced, cracked infills subjected to transverse forces shall be assumed to be equal to zero unless arching action is considered.

Arching action shall be considered if, and only if, the following conditions exist.

- The panel is in tight contact with the surrounding frame components.
- The product of the elastic modulus, E_{fe}, times the moment of inertia, I_f, of the most flexible frame component exceeds a value of 3.6 x 10^9 lb-in.^{2}.
- The frame components have sufficient strength to resist thrusts from arching of an infill panel.
- The h_{inf}/t_{inf} ratio is less than or equal to 25.

For such cases, mid-height deflection normal to the plane of an infill panel, Δ_{inf}, divided by the infill height, h_{inf}, shall be determined in accordance with Equation 7-20.

$$\frac{\Delta_{inf}}{h_{inf}} = \frac{0.002\left(\frac{h_{inf}}{t_{inf}}\right)}{1 + \sqrt{1 - 0.002\left(\frac{h_{inf}}{t_{inf}}\right)^2}} \qquad (7\text{-}20)$$

For infill panels not meeting the requirements for arching action, deflections shall be determined with the procedures given in Sections 7.4.3 or 7.4.5.

Stiffnesses for existing and new infills shall be assumed to be the same.

7.5.3.2 Strength Acceptability Criteria

Masonry infill panels shall resist out-of-plane inertial forces as given in Section 2.11.7. Transversely loaded infills shall not be analyzed with the Linear or Nonlinear Static Procedures prescribed in Chapter 3.

The lower bound transverse strength of a URM infill panel shall exceed normal pressures as prescribed in Section 2.11.7. Unless arching action is considered, the lower bound strength of a URM infill panel shall be limited by the lower bound masonry flexural tension strength, f'_t, which may be taken as 0.7 times the expected tensile strength, f_{te}, as determined per Section 7.3.2.3.

Arching action shall be considered only when the requirements stated in the previous section are met. In such case, the lower bound transverse strength of an infill panel in pounds per square foot, q_{in}, shall be determined using Equation 7-21.

$$Q_{CL} = q_{in} = \frac{0.7 f'_m \lambda_2}{\left(\frac{h_{inf}}{t_{inf}}\right)} \times 144 \qquad (7\text{-}21)$$

where

f'_m = Lower bound of masonry compressive strength equal to $f_{me}/1.6$, psi

λ_2 = Slenderness parameter as defined in Table 7-9

Table 7-9 ***Values of λ_2 for Use in Equation 7-21***

h_{inf}/t_{inf}	5	10	15	25
λ_2	0.129	0.060	0.034	0.013

7.5.3.3 Deformation Acceptance Criteria

The Immediate Occupancy Performance Level shall be met when significant visual cracking of a URM infill occurs. This limit state shall be assumed to occur at an out-of-plane drift of a story panel equal to approximately 2%.

The Life Safety Performance Level shall be met when substantial damage to a URM infill occurs, and the potential for the panel, or some portion of it, to drop out of the frame is high. This limit state shall be assumed to occur at an out-of-plane lateral story drift ratio equal to approximately 3%.

If the surrounding frame can remain stable following the loss of an infill panel, infill panels shall not be subject to limits set by the Collapse Prevention Performance Level.

Acceptable deformations of existing and new walls shall be assumed to be the same.

7.6 Anchorage to Masonry Walls

7.6.1 Types of Anchors

Anchors considered in Section 7.6.2 shall include plate anchors, headed anchor bolts, and bent bar anchor bolts embedded into clay-unit and concrete masonry. Anchors in hollow-unit masonry must be embedded in grout.

Pullout and shear strength of expansion anchors shall be verified by tests.

7.6.2 Analysis of Anchors

Anchors embedded into existing or new masonry walls shall be considered as force-controlled components. Lower bound values for strengths with respect to pullout, shear, and combinations of pullout and shear, shall be based on Section 8.3.12 of BSSC (1995) for embedded anchors.

The minimum effective embedment length for considerations of pullout strength shall be as defined in Section 8.3.12 of BSSC (1995). When the embedment length is less than four bolt diameters or two inches (50.8 mm), the pullout strength shall be taken as zero.

The minimum edge distance for considerations of full shear strength shall be 12 bar diameters. Shear strength of anchors with edge distances equal to or less than one inch (25.4 mm) shall be taken as zero. Linear interpolation of shear strength for edge distances between these two bounds is permitted.

7.7 Masonry Foundation Elements

7.7.1 Types of Masonry Foundations

Masonry foundations are common in older buildings and are still used for some modern construction. Such foundations may include footings and foundation walls constructed of stone, clay brick, or concrete block. Generally, masonry footings are unreinforced; foundation walls may or may not be reinforced.

Spread footings transmit vertical column and wall loads to the soil by direct bearing. Lateral forces are transferred through friction between the soil and the masonry, as well as by passive pressure of the soil acting on the vertical face of the footing.

7.7.2 Analysis of Existing Foundations

A lateral-force analysis of a building system shall include the deformability of the masonry footings, and the flexibility of the soil under them. The strength and stiffness of the soil shall be checked per Section 4.4.

Masonry footings shall be modeled as elastic components with little or no inelastic deformation capacity, unless verification tests are done to prove otherwise. For the Linear Static Procedure, masonry footings shall be considered to be force-controlled components (m equals 1.0).

Masonry retaining walls shall resist active and passive soil pressures per Section 4.5. Stiffness, and strength and acceptability criteria for masonry retaining walls shall be the same as for other masonry walls subjected to transverse loadings, as addressed in Sections 7.4.3 and 7.4.5.

7.7.3 Rehabilitation Measures

In addition to those rehabilitation measures provided for concrete foundation elements in Section 6.13.4, masonry foundation elements may also be rehabilitated with the following options:

- Injection grouting of stone foundations
- Reinforcing of URM foundations

- Prestressing of masonry foundations
- Enlargement of footings by placement of reinforced shotcrete
- Enlargement of footings with additional reinforced concrete sections

Procedures for rehabilitation shall follow provisions for enhancement of masonry walls where applicable per Section 7.4.1.3.

7.8 Definitions

Bearing wall: A wall that supports gravity loads of at least 200 pounds per linear foot from floors and/or roofs.

Bed joint: The horizontal layer of mortar on which a masonry unit is laid.

Cavity wall: A masonry wall with an air space between wythes. Wythes are usually joined by wire reinforcement, or steel ties. Also known as a noncomposite wall.

Clay-unit masonry: Masonry constructed with solid, cored, or hollow units made of clay. Hollow clay units may be ungrouted, or grouted.

Clay tile masonry: Masonry constructed with hollow units made of clay tile. Typically, units are laid with cells running horizontally, and are thus ungrouted. In some cases, units are placed with cells running vertically, and may or may not be grouted.

Collar joint: Vertical longitudinal joint between wythes of masonry or between masonry wythe and back-up construction that may be filled with mortar or grout.

Composite masonry wall: Multiwythe masonry wall acting with composite action.

Concrete masonry: Masonry constructed with solid or hollow units made of concrete. Hollow concrete units may be ungrouted, or grouted.

Head joint: Vertical mortar joint placed between masonry units in the same wythe.

Hollow masonry unit: A masonry unit whose net cross-sectional area in every plane parallel to the bearing surface is less than 75% of the gross cross-sectional area in the same plane.

Infill: A panel of masonry placed within a steel or concrete frame. Panels separated from the surrounding frame by a gap are termed “isolated infills.” Panels that are in tight contact with a frame around its full perimeter are termed “shear infills.”

In-plane wall: See shear wall.

Masonry: The assemblage of masonry units, mortar, and possibly grout and/or reinforcement. Types of masonry are classified herein with respect to the type of the masonry units, such as clay-unit masonry, concrete masonry, or hollow-clay tile masonry.

Nonbearing wall: A wall that supports gravity loads less than as defined for a bearing wall.

Noncomposite masonry wall: Multiwythe masonry wall acting without composite action.

Out-of-plane wall: A wall that resists lateral forces applied normal to its plane.

Parapet: Portions of a wall extending above the roof diaphragm. Parapets can be considered as flanges to roof diaphragms if adequate connections exist or are provided.

Partially grouted masonry wall: A masonry wall containing grout in some of the cells.

Perforated wall or infill panel: A wall or panel not meeting the requirements for a solid wall or infill panel.

Pier: A vertical portion of masonry wall between two horizontally adjacent openings. Piers resist axial stresses from gravity forces, and bending moments from combined gravity and lateral forces.

Reinforced masonry (RM) wall: A masonry wall that is reinforced in both the vertical and horizontal directions. The sum of the areas of horizontal and vertical reinforcement must be at least 0.002 times the gross cross-sectional area of the wall, and the minimum area of reinforcement in each direction must be not less than 0.0007 times the gross cross-sectional area of the wall. Reinforced walls are assumed to resist loads through resistance of the masonry in compression and the reinforcing steel in tension or compression. Reinforced masonry is partially grouted or fully grouted.

Running bond: A pattern of masonry where the head joints are staggered between adjacent courses by more than a third of the length of a masonry unit. Also refers to the placement of masonry units such that head joints in successive courses are horizontally offset at least one-quarter the unit length.

Shear wall: A wall that resists lateral forces applied parallel with its plane. Also known as an in-plane wall.

Solid masonry unit: A masonry unit whose net cross-sectional area in every plane parallel to the bearing surface is 75% or more of the gross cross-sectional area in the same plane.

Solid wall or solid infill panel: A wall or infill panel with openings not exceeding 5% of the wall surface area. The maximum length or height of an opening in a solid wall must not exceed 10% of the wall width or story height. Openings in a solid wall or infill panel must be located within the middle 50% of a wall length and story height, and must not be contiguous with adjacent openings.

Stack bond: In contrast to running bond, usually a placement of units such that the head joints in successive courses are aligned vertically.

Transverse wall: A wall that is oriented transverse to the in-plane shear walls, and resists lateral forces applied normal to its plane. Also known as an out-of-plane wall.

Unreinforced masonry (URM) wall: A masonry wall containing less than the minimum amounts of reinforcement as defined for masonry (RM) walls. An unreinforced wall is assumed to resist gravity and lateral loads solely through resistance of the masonry materials.

Wythe: A continuous vertical section of a wall, one masonry unit in thickness.

7.9 Symbols

A_b Sum of net mortared area of bed joints above and below test unit, in.^{2}

A_{es} Area of equivalent strut for masonry infill, in.^{2}

A_n Area of net mortared/grouted section of wall or pier, in.^{2}

A_{ni} Area of net mortared/grouted section of masonry infill, in.^{2}

A_s Area of reinforcement, in.^{2}

E_{fe} Expected elastic modulus of frame material, psi

E_{me} Elastic modulus of masonry in compression as determined per Section 7.3.2.2, psi

E_{se} Expected elastic modulus of reinforcing steel per Section 7.3.2.6, psi

G_{me} Shear modulus of masonry as determined per Section 7.3.2.5, psi

I Moment of inertia of section, in.^{4}

I_{col} Moment of inertia of column section, in.^{4}

I_f Moment of inertia of smallest frame member confining infill panel, in.^{4}

L Length of wall, in.

L_{inf} Length of infill panel, in.

M Moment on masonry section, in.-lb

M/V Ratio of expected moment to shear acting on wall or pier

P_c Lower bound of vertical compressive strength for wall or pier, lb

P_{CE} Expected vertical axial compressive force for load combinations in Equations 3-14 and 3-15, lb

P_{CL} Lower bound of vertical compressive force for load combination of Equation 3-3, lb

Q_{CE} Expected strength of a component or element at the deformation level under consideration in a deformation-controlled action

Q_{CL} Lower bound estimate of the strength of a component or element at the deformation level under consideration for a force-controlled action

Q_E Unreduced earthquake demand forces used in Equation 3-14

Q_G Gravity force acting on component as defined in Section 3.2.8

V Shear on masonry section, lb

V_{bjs} Expected shear strength of wall or pier based on bed-joint shear stress, lb

V_{CL} Lower bound shear capacity, lb

V_{dt} Lower bound of shear strength based on diagonal tension for wall or pier, lb

V_{fre} Expected story shear strength of bare frame, lb

V_{ine} Expected shear strength of infill panel, lb

V_{mL} Lower bound shear strength provided by masonry, lb

V_r Expected shear strength of wall or pier based on rocking, lb

V_{sL} Lower bound shear strength provided by shear reinforcement, lb

V_{tc} Lower bound of shear strength based on toe compressive stress for wall or pier, lb

V_{test} Measured force at first movement of a masonry unit with in-place shear test, lb

a Equivalent width of infill strut, in.

c Fraction of strength loss for secondary elements as defined in Figure 7-1

d Wall, pier, or infill inelastic drift percentage as defined in Figure 7-1

d_v Length of component in direction of shear force, in.

e Wall, pier, or infill inelastic drift percentage as defined in Figure 7-1

f_a Lower bound of vertical compressive stress, psi

f_{ae} Expected vertical compressive stress, psi

f_{me} Expected compressive strength of masonry as determined in Section 7.3.2.1, psi

f_{te} Expected masonry flexural tensile strength as determined in Section 7.3.2.3, psi

f_{vie} Expected shear strength of masonry infill, psi

f_y Lower bound of yield strength of reinforcing steel, psi

f_{ye} Expected yield strength of reinforcing steel as determined in Section 7.3.2.6, psi

f'_{dt} Lower bound of masonry diagonal tension strength, psi

f'_m Lower bound of masonry compressive strength, psi

f'_t Lower bound masonry tensile strength, psi

h Height of a column, pilaster, or wall, in.

h_{col} Height of column between beam centerlines, in.

h_{eff} Height to resultant of lateral force for wall or pier, in.

h_{inf} Height of infill panel, in.

k Lateral stiffness of wall or pier, lb/in.

l_{beff} Assumed distance to infill strut reaction point for beams as shown in Figure C7-5, in.

l_{ceff} Assumed distance to infill strut reaction point for columns as shown in Figure C7-4, in.

p_{D+L} Expected gravity stress at test location, psi

q_{ine} Expected transverse strength of an infill panel, psf

r_{inf} Diagonal length of infill panel, in.

t Least thickness of wall or pier, in.

t_{inf} Thickness of infill panel, in.

v_{me} Expected masonry shear strength as determined by Equation 7-1, psi

v_{te} Average bed-joint shear strength, psi

v_{to} Bed-joint shear stress from single test, psi

Δ_{inf} Deflection of infill panel at mid-length when subjected to transverse loads, in.

α Factor equal to 0.5 for fixed-free cantilevered shear wall, or 1.0 for fixed-fixed pier

β Ratio of expected frame strength to expected infill strength

θ Angle between infill diagonal and horizontal axis, $\tan \theta = L_{inf}/h_{inf}$, radians

θ_b Angle between lower edge of compressive strut and beam as shown in Figure C7-5, radians

θ_c Angle between lower edge of compressive strut and beam as shown in Figure C7-4, radians

κ A reliability coefficient used to reduce component strength values for existing components based on the quality of knowledge about the components' properties (see Section 2.7.2)

λ_1 Coefficient used to determine equivalent width of infill strut

λ_2 Infill slenderness factor

ρ_g Ratio of area of total wall or pier reinforcement to area of gross section

7.10 References

ASTM, latest editions, Standards having the numbers A615, E22, E518, American Society for Testing Materials, Philadelphia, Pennsylvania.

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Masonry Standards Joint Committee (MSJC), 1995a, *Building Code Requirements for Masonry Structures*, ACI 530-95/ASCE 5-95/TMS 402-95, American Concrete Institute, Detroit, Michigan; American Society of Civil Engineers, New York, New York; and The Masonry Society, Boulder, Colorado.

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8. Wood and Light Metal Framing (Systematic Rehabilitation)

8.1 Scope

This chapter presents the general methods for rehabilitating wood frame buildings and/or wood and light metal framed elements of other types of buildings that are presented in other sections of this document. Refer to Chapter 2 for the general methodology and issues regarding performance goals, and the decisions and steps necessary for the engineer to develop a rehabilitation scheme. The Linear Static Procedure (LSP) presented in Chapter 3 is the method recommended for the systematic analysis of wood frame buildings. However, properties of the idealized elastic and inelastic performance of the various elements and connections are included so that nonlinear procedures can be used if desired.

A general history of the development of wood framing methods is presented in Section 8.2, along with the features likely to be found in buildings of different ages. A more complete historical perspective is included in the *Commentary*. The evaluation and assessment of various structural elements of wood frame buildings is found in Section 8.3. For a description and discussion of connections between the various components and elements, see Section 8.3.2.2B. Properties of shear walls and other lateral-force-resisting systems such as braced frames are described and discussed in Section 8.4, along with various retrofit or strengthening methods. Horizontal floor and roof diaphragms and braced systems are discussed in Section 8.5, which also covers engineering properties and methods of upgrading or strengthening the elements. Wood foundations and pole structures are described in Section 8.6. For additional information regarding foundations, see Chapter 4. Definitions of terms are in Section 8.7; symbols are in Section 8.8. Reference materials for both new and existing materials are provided in Section 8.9.

8.2 Historical Perspective

8.2.1 General

Wood frame construction has evolved over millennia; wood is the primary building material of most residential and small commercial structures in the United States. It has often been used for the framing of roofs and/or floors, in combination with other materials, for other types of buildings.

Establishing the age and recognizing the location of a building can be helpful in determining what types of lateral-force-resisting systems may be present. Information regarding the establishment of a building's age and a discussion of the evolution of framing systems can be found in Section C8.2 of the *Commentary*.

As indicated in Chapter 1, great care should be exercised in selecting the appropriate rehabilitation approaches and techniques for application to historic buildings in order to preserve their unique characteristics.

8.2.2 Building Age

Based on the approximate age of a building, various assumptions can be made about the design and features of construction. Older wood frame structures that predate building codes and standards usually do not have the types of elements considered essential for predictable seismic performance. These elements will generally have to be added, or the existing elements upgraded by the addition of lateral-load-resisting components to the existing structure, in order to obtain a predictable performance.

If the age of a building is known, the code in effect at the time of construction and the general quality of the construction usual for the time can be helpful in evaluating an existing building. The level of maintenance of a building may be a helpful guide in determining the extent of loss of a structure's capacity to resist loads.

8.2.3 Evolution of Framing Methods

The earliest wood frame buildings built by European immigrants to the United States were built with post and beam or frame construction adopted from Europe and the British Isles. This was followed by the development of balloon framing in about 1830 in the Midwest, which spread to the East Coast by the 1860s. This, in turn, was followed by the development of western or platform framing shortly after the turn of the century. Platform framing is the system currently in use for multistory construction.

Drywall or wallboard was first introduced in about 1920; however, its use was not widespread until after World War II, when gypsum lath (button board) also came into extensive use as a replacement for wood lath.

With the exception of public schools in high seismic areas, modern wood frame structures detailed to resist seismic loads were generally not built prior to 1934. For most wood frame structures, either general seismic provisions were not provided—or the codes that included them were not enforced—until the mid-1950s or later, even in the most active seismic areas. (This time frame varies somewhat depending on local conditions and practice.)

Buildings constructed after 1970 in high seismic areas usually included a well-defined lateral-force-resisting system as a part of the design. However, site inspections and code enforcement varied greatly, so that the inclusion of various features and details on the plans does not necessarily mean that they are in place or fully effective. Verification is needed to ensure that good construction practices were followed.

Until about 1950, wood residential buildings were frequently constructed on raised foundations and in some cases included a short stud wall, called a “cripple wall,” between the foundation and the first floor framing. This occurs on both balloon framed and platform framed buildings. There may be an extra demand on these cripple walls, because most interior partition walls do not continue to the foundation. Special attention is required for these situations. Adequate bracing must be provided for cripple walls as well as the attachment of the sill plate to the foundation.

In more recent times, light gage metal studs and joists have been used in lieu of wood framing for some structures. Lateral-load resistance is either provided by metal straps attached to the studs and top and bottom tracks, or by structural panels attached with sheet metal screws to the studs and the top and bottom track in a manner similar to wood construction. The metal studs and joists vary in size, gage, and configuration depending on the manufacturer and the loading conditions.

For systems using structural panels for bracing, see Section 8.4 for analysis and acceptance criteria. For the all-metal systems using steel strap braces, see Chapter 5 for guidance.

8.3 Material Properties and Condition Assessment

8.3.1 General

Each structural element in an existing building is composed of a material capable of resisting and transferring applied loads to the foundation systems. One material group historically used in building construction is wood. Various grades and species of wood have been used in a cut dimension form, combined with other structural materials (e.g., steel/wood elements), or in multiple layers of construction (e.g., glued-laminated wood components). Wood materials have also been manufactured into hardboard, plywood, and particleboard products, which may have structural or nonstructural functions in construction. The condition of the in-place wood materials will greatly influence the future behavior of wood components in the building system.

Quantification of in-place material properties and verification of existing system configuration and condition are necessary to properly analyze the building. The focus of this effort shall be given to the primary vertical- and lateral-load-resisting elements and components thereof. These primary components may be identified through initial analysis and application of loads to the building model.

The extent of in-place materials testing and condition assessment that must be accomplished is related to availability and accuracy of construction documents and as-built records, the quality of materials used and construction performed, and physical condition. A specific problem with wood construction is that structural wood components are often covered with other components, materials, or finishes; in addition, their behavior is influenced by past loading history. Knowledge of the properties and grades of material used in original component/connection fabrication is invaluable, and may be effectively used to reduce the amount of in-place testing required. The design professional is encouraged to research and acquire all available records from original construction, including design calculations.

Connection configuration also has a very important influence on response to applied loads and motions. A large number of connector types exist, the most prevalent being nails and through-bolts. However, more recent construction has included metal straps and hangers, clip angles, and truss plates. An understanding of connector configuration and mechanical properties must be gained to analyze properly the anticipated performance of the building construction.

8.3.2 Properties of In-Place Materials and Components

8.3.2.1 Material Properties

Wood has dramatically different properties in its three orthotropic axes (parallel to grain, transverse to grain, and radial). These properties vary with wood species, grade, and density. The type, grade, and condition of the component(s) must be established in order to compute strength and deformation characteristics. Mechanical properties and configuration of component and connection material dictate the structural behavior of the component under load. The effort required to determine these properties is related to the availability of original and updated construction documents, original quality of construction, accessibility, condition of materials, and the analytical procedure to be used in the rehabilitation (e.g., LSP for wood construction).

The first step in quantifying properties is to establish the species and grade of wood through review of construction documents or direct inspection. If the wood is not easily identified visually or by the presence of a stamped grade mark on the wood surface, then samples can be taken for laboratory testing and identification (see below). The grade of the wood also may be established from grade marks, the size and presence of knots, splits and checks, the slope of the grain, and the spacing of growth rings through the use of appropriate grade rules. The grading shall be performed using a specific grading handbook for the assumed wood species and application (e.g., Department of Commerce American Softwood Lumber Standard PS 20-70 (NIST, 1986), the National Grading Rules for Dimension Lumber of the National Grading Rules Committee), or through the use of the ASTM (1992) D245 grading methodology.

In general, the determination of material species and properties (other than inter-component connection behavior) is best accomplished through removal of samples coupled with laboratory analysis by experts in wood science. Sampling shall take place in regions of reduced stress, such as mid-depth of members. Some local repair may be necessary after sampling.

The properties of adhesives used in fabrication of certain component types (e.g., laminated products) must also be evaluated. Such adhesives may be adversely affected by exposure to moisture and other conditions in-service. Material properties may also be affected by certain chemical treatments (e.g., fire retardant) originally applied to protect the component from environmental conditions.

8.3.2.2 Component Properties

A. Elements

Structural elements of the lateral-force-resisting system comprise primary and secondary components, which collectively define element strength and resistance to deformation. Behavior of the components—including shear walls, beams, diaphragms, columns, and braces—is dictated by physical properties such as area; material grade; thickness, depth, and slenderness ratios; lateral torsional buckling resistance; and connection details. The following component properties shall be established during a condition assessment in the initial stages of the seismic rehabilitation process, to aid in evaluating component strength and deformation capacities (see Section 8.3.3 for assessment guidelines):

- Original cross sectional shape and physical dimensions (e.g., actual dimensions for 2" x 4" stud) for the primary members of the structure
- Size and thickness of additional connected materials, including plywood, bracing, stiffeners; chord, sills, struts, and hold-down posts
- Modifications to members (e.g., notching, holes, splits, cracks)
- Location and dimension of braced frames and shear walls; type, grade, nail size, and spacing of hold-downs and drag/strut members
- Current physical condition of members, including presence of decay or deformation
- Confirmation of component(s) behavior with overall element behavior

These primary component properties are needed to properly characterize building performance in the seismic analysis. The starting point for establishing component properties should be the available construction documents. Preliminary review of these documents shall be performed to identify primary vertical- (gravity-) and lateral-load-carrying elements and systems, and their critical components and connections. Site inspections should be conducted to verify conditions and to assure that remodeling has not changed the original design concept. In the absence of a complete set of building drawings, the design professional must perform a thorough inspection of the building to identify these elements, systems, and components as indicated in Section 8.3.3. Where reliable record drawings do not exist, an as-built set of plans for the building must be created.

B. Connections

The method of connecting the various elements of the structural system is critical to its performance. The type and character of the connections must be determined by a review of the plans and a field verification of the conditions.

The following connections shall be established during a condition assessment to aid in the evaluation of the structural behavior:

- Connections between horizontal diaphragms and shear walls and braced frames
- Size and character of all drag ties and struts, including splice connections used to collect loads from the diaphragms to deliver to shear walls or braced frames
- Connections at splices in chord members of horizontal diaphragms
- Connections of horizontal diaphragms to exterior or interior concrete or masonry walls for both in-plane and out-of-plane loads
- Connections of cross-tie members for concrete or masonry buildings
- Connections of shear walls to foundations
- Method of through-floor transfer of wall shears in multistory buildings

8.3.2.3 Test Methods to Quantify Properties

To obtain the desired in-place mechanical properties of materials and components, including expected strength, it is often necessary to use proven destructive and nondestructive testing methods.

Of greatest interest to wood building system performance are the expected orthotropic strengths of the installed materials for anticipated actions (e.g., flexure). Past research and accumulation of data by industry groups have led to published mechanical properties for most wood types and sizes (e.g., dimensional solid-sawn lumber, and glued-laminated or “glulam” beams). Section 8.3.2.5 addresses these established default strengths and distortion properties. This information may be used, together with tests from recovered samples, to establish rapidly the expected strength properties for use in component strength and deformation analyses. Where possible, the load history for the building shall be assessed for possible influence on component strength and deformation properties.

To quantify material properties and analyze the performance of archaic wood construction, shear walls, and diaphragm action, more extensive sampling and testing may be necessary. This testing should include further evaluation of load history and moisture effects on properties, and the examination of wall and diaphragm continuity, and suitability of in-place connectors.

Where it is desired to use an existing assembly and little or no information as to its performance is available, a cyclic load test of a mock-up of the existing structural elements can be utilized to determine the performance of various assemblies, connections, and load transfer conditions. The establishment of the parameters given in Tables 8-1 and 8-2 can be determined from the results of the cyclic load tests. See Section 2.13 for an explanation of the backbone curve and the establishment of parameters.

8.3.2.4 Minimum Number of Tests

In order to accurately quantify expected strength and other in-place properties, it is important that a minimum number of tests be conducted on representative components. The minimum number of tests is dictated by available data from original construction, the type of structural system employed, desired accuracy, and quality/condition of in-place materials. Visual access to the structural system also influences testing program definition. As an alternative, the design professional may elect to utilize the default strength properties, per Section 8.3.2.5 provisions, as opposed to conducting the specified testing. However, these default values should only be used for the LSP. It is strongly encouraged that the expected strengths be derived through testing of assemblies to model behavior more accurately.

In terms of defining expected strength properties, the following guidelines should be followed. Removal of coverings, including stucco, fireproofing and partition materials, is generally required to facilitate the sampling and observations.

- If original construction documents exist that define the grade of wood and mechanical properties, at least one location for each story shall be randomly observed from each component type (e.g., solid sawn lumber, glulam beam, plywood diaphragm) identified as having a different material grade. These shall be verified by sampling and testing or by observing grade stamps and conditions.

- If original construction documents defining properties are limited or do not exist, but the date of construction is known and single material use is confirmed (e.g., all components are Douglas fir solid sawn lumber), at least three observations or samples should be randomly made for each component type (e.g., beam, column, shear wall) for every two floors in the building.

- If no knowledge of the structural system and materials used exists, at least six samples shall be removed or observed from each element (e.g., primary gravity- and lateral-load-resisting components) and component type (e.g., solid sawn lumber, diaphragm) for every two floors of construction. If it is determined from testing and/or observation that more than one material grade exists, additional observations should be made until the extent of use for each grade in component fabrication has been established.

- In the absence of construction records defining connector features present, at least three connectors shall be observed for every floor of the building. The observations shall consist of each connector type present in the building (e.g., nails, bolts, straps), such that the composite strength of the connection can be estimated.

- For an archaic systems test or other full-scale mock-up test of an assembly, at least two cyclic tests of each assembly shall be conducted. A third test shall be conducted if the results of the two tests vary by more than 20%.

8.3.2.5 Default Properties

Mechanical properties for wood materials and components are based on available historical data and tests on samples of components, or mock-up tests of typical systems. In the absence of these data, or for comparative purposes, default material strength properties are needed. Unlike other structural materials, default properties for wood are highly variable and dependent on factors including the species, grade, usage, age, and exposure conditions. As a minimum, it is recommended that the type and grade of wood be established. Historically, codes and standards including the National Design Specification (AF&PA, 1991a) have published allowable stresses as opposed to strengths. These values are conservative, representing mean values from previous research. Default strength values, consistent with *NEHRP Recommended Provisions* (BSSC, 1995), may be calculated as the allowable stress multiplied by a 2.16 conversion factor, a capacity reduction factor of 0.8, and time effect factor of 1.6 for seismic loading. This results in an approximate 2.8 factor to translate allowable stress values to yield or limit state values. The expected strength, Q_{CE}, is determined based on these yield or limit state strengths. If significant inherent damage or deterioration is found to be present, default values may not be used. Structural elements with significant damage need to be replaced with new materials, or else a significant reduction in the capacity and stiffness must be incorporated into the analysis.

It is recommended that the results of any material testing performed be compared to the default values for the particular era of building construction; should significantly reduced properties from testing be discovered in this testing, further evaluation as to the cause shall be undertaken. Default values may not be used if they are greater than those obtained from testing.

Default material strength properties may only be used in conjunction with the LSP. For all other analysis procedures, expected strengths from specified testing and/or mock-up testing shall be used to determine anticipated performance.

Default values for connectors shall be established in a manner similar to that for the members. The published values in the National Design Specification (AF&PA,

1994) shall be increased by a factor of 2.8 to convert from allowable stress levels to yield or limit state values, Q_{CE}, for seismic loading.

Deformation at yield of nailed connectors may be assumed to be 0.02 inches for wood to wood and wood to metal connections. For wood screws the deformation may be assumed to be 0.05 inches; for lag bolts the deformation may be assumed at 0.10 inches. For bolts, the deformation for wood to wood connections can be assumed to be 0.2 inches; for wood to steel connections, 0.15 inches. In addition the estimated deformation of any hardware or allowance, e.g., for poor fit or oversized holes, should be summed to obtain the total connection deformation.

8.3.3 Condition Assessment

8.3.3.1 General

A condition assessment of the existing building and site conditions shall be performed as part of the seismic rehabilitation process. The goal of this assessment is fourfold:

1. To examine the physical condition of primary and secondary components and the presence of any degradation
2. To verify the presence and configuration of components and their connections, and continuity of load paths between components, elements, and systems
3. To review other conditions—such as neighboring party walls and buildings, presence of nonstructural components, prior remodeling, and limitations for rehabilitation—that may influence building performance
4. To formulate a basis for selecting a knowledge factor (see Section 8.3.4)

The physical condition of existing components and elements, and their connections, must be examined for presence of degradation. Degradation may include environmental effects (e.g., decay; splitting; fire damage; biological, termite, and chemical attack) or past/current loading effects (e.g., overload, damage from past earthquakes, crushing, and twisting). Natural wood also has inherent discontinuities such as knots, checks, and splits that must be accounted for. The condition assessment shall also examine for configuration problems observed in recent earthquakes, including effects of discontinuous components, improper nailing or bolting, poor fit-up, and connection problems at the foundation level. Often, unfinished areas such as attic spaces, basements, and crawl spaces provide suitable access to wood components used and can give a general indication of the condition of the rest of the structure. Invasive inspection of critical components and connections is typically required.

Connections in wood components, elements, and systems require special consideration and evaluation. The load path for the system must be determined, and each connection in the load path(s) must be evaluated. This includes diaphragm-to-component and component-to-component connections. The strength and deformation capacity of connections must be checked where the connection is attached to one or more components that are expected to experience significant inelastic response. Anchorage of exterior walls to roof and floors for concrete and masonry buildings, for which wood diaphragms are used for out-of-plane loading, requires detailed inspection. Bolt holes in relatively narrow straps sometime preclude the ductile behavior of the steel strap. Twists and kinks in the strap can also have a serious impact on its anticipated behavior. Cross-ties across the building, which are part of the wall anchorage system, need to be inspected to confirm their presence and the connection of each piece, to ensure that a positive load path exists to tie the building walls together.

The condition assessment also affords an opportunity to review other conditions that may influence wood elements and systems, and overall building performance. Of particular importance is the identification of other elements and components that may contribute to or impair the performance of the wood system in question, including infills, neighboring buildings, and equipment attachments. Limitations posed by existing coverings, wall and ceiling space insulation, and other material shall also be defined such that prudent rehabilitation measures can be planned.

8.3.3.2 Scope and Procedures

The scope of a condition assessment shall include all primary structural elements and components involved in gravity- and lateral-load resistance. Accessibility constraints may necessitate the use of instruments such as a fiberscope or video probe to reduce the amount of damage to covering materials and fabrics. The knowledge and insight gained from the condition assessment is invaluable to the understanding of load paths and the ability of components to resist and transfer these loads. The degree of assessment performed also affects the knowledge factor, κ, discussed in Section 8.3.4. General guidelines and procedures are also contained in that section; for additional guidance and references, see the *Commentary.*

Direct visual inspection provides the most valuable information, as it can be used to quickly identify any configuration issues, and allows both measurement of component dimensions, and determination whether degradation is present. The continuity of load paths may be established through viewing of components and connection condition. From visual inspection, the need for other test methods to quantify the presence and degree of degradation may be established.

The dimensions and features of all accessible components shall be measured and compared to available design information. Similarly, the configuration and condition of all connections shall be verified, with any deformations or other anomalies noted. If design documents for the structure do not exist, this technique shall be followed to develop an as-built drawing set.

If coverings or other obstructions exist, indirect visual inspection through use of drilled holes and a fiberscope shall be utilized (as allowed by access). If this method is not appropriate, then local removal of covering materials will be necessary. The following guidelines shall be used.

- If detailed design drawings exist, exposure of at least three different primary connections shall occur for each connection type (e.g., beam-column, shear wall-diaphragm, shear wall-foundation). If no significant capacity-reducing deviations from the drawings exist, the sample may be considered representative. If deviations are noted, then removal of all coverings from primary connections of that type may be necessary, if reliance is to be placed on the connection.

- In the absence of accurate drawings, either invasive fiberscopic inspections or exposure of at least 50% of all primary connection types for inspection shall occur. If common detailing is observed, this sample may be considered representative. If a multitude of details or conditions are observed, full exposure is required.

The scope of this removal effort is dictated by the component and element design. For example, in a braced frame, exposure of several key connections may suffice if the physical condition is acceptable and configuration matches the design drawings. However, for shear walls and diaphragms it may be necessary to expose more connection points because of varying designs and the critical nature of the connections. For encased walls and frames for which no drawings exist, it is necessary to indirectly view or expose all primary end connections for verification.

Physical condition of components and connectors may also support the need to use certain destructive and nondestructive test methods. Devices normally utilized for the detection of reinforcing steel in concrete or masonry can be utilized to verify the extent of metal straps and hardware located beneath the finish surfaces. Further guidelines and procedures for destructive and nondestructive tests that may be used in the condition assessment are contained in the *Commentary.*

8.3.3.3 Quantifying Results

The results of the condition assessment shall be used in the preparation of building system models for evaluation of seismic performance. To aid in this effort, the results shall be quantified and reduced, with the following specific topics addressed:

- Component section properties and dimensions

- Component configuration and presence of any eccentricities

- Interaction of nonstructural components and their involvement in lateral-load resistance

The acceptance criteria for existing components depend on the design professional’s knowledge of the condition of the structural system and material properties, as previously noted. Certain damage—such as water staining, evidence of prior leakage, splitting, cracking, checking, warping, and twisting—may be allowable. The design professional must establish a case-by-case acceptance for such damage on the basis of capacity loss or deformation constraints. Degradation at connection points should be carefully examined; significant capacity reductions may be involved, as well as a loss of ductility. All deviations noted between available construction records and as-built conditions shall be accounted for and considered in the structural analysis.

8.3.4 Knowledge (κ) Factor

As described in Section 2.7, computation of component capacities and allowable deformations shall involve the use of a knowledge (κ) factor. For cases where an LSP will be used in the analysis, two categories of κ exist. This section further describes those requirements specific to wood structural elements that must be accomplished in the selection of a κ factor.

If the wood structural system is exposed, significant knowledge regarding configuration and behavior may be gained through condition assessment. In general, a κ factor of 1.0 can be utilized when a thorough assessment is performed on the primary and secondary components and load path, and the requirements of Section 2.7 are met. Similarly, if the wood system is encased, a κ factor of 1.0 can be utilized when three samples of each primary component connection type are exposed and verified as being compliant with construction records, and fiberscopic examinations are performed to confirm the condition and configuration of primary load-resisting components.

If knowledge of as-built component or connection configuration/condition is incomplete or nonexistent, the κ factor used in the final component evaluation shall be reduced to 0.75. Examples of where this value shall be applied are contained in Section 2.7 and Equation 3-18. For encased components where construction documents are limited and knowledge of configuration and condition is incomplete, a factor of 0.75 shall be used.

8.3.5 Rehabilitation Issues

Upon determining that portions of a wood building structure are deficient or inadequate for the Rehabilitation Objective, the next step is to define reinforcement or replacement alternatives. If a reinforcement program is to be followed and attachment to the existing framing system is proposed, it is necessary to closely examine material factors that may influence reinforcement/attachment design, including:

- Degree of any degradation in the component from such mechanisms as biological attack, creep, high static or dynamic loading, moisture, or other effects
- Level of steady state stress in the components to be reinforced (and potential to temporarily remove this stress if appropriate)
- Elastic and plastic properties of existing components, to preserve strain compatibility with any new reinforcement materials
- Ductility, durability, and suitability of existing connectors between components, and access for reinforcement or modification
- Prerequisite efforts necessary to achieve appropriate fit-up for reinforcing components and connections
- Load flow and deformation of the components at end connections (especially at foundation attachments and connections where mixed connectors such as bolts and nails exist)
- Presence of components manufactured with archaic materials, which may contain material discontinuities and shall be examined during the rehabilitation design to ensure that the selected reinforcement is feasible

8.4 Wood and Light Frame Shear Walls

The behavior of wood and light frame shear walls is complex and influenced by many factors, the primary factor being the wall sheathing. Wall sheathings can be divided into many categories (e.g., brittle, elastic, strong, weak, good at dissipating energy, and poor at dissipating energy). In many existing buildings, the walls were not expected to act as shear walls (e.g., a wall sheathed with wood lath and plaster). Most shear walls are designed based on values from monotonic load tests and historically accepted values. The allowable shear per unit length used for design was assumed to be the same for long walls, narrow walls, walls with stiff tie-downs, and walls with flexible tie-downs. Only recently have shear wall assemblies (framing, covering, anchorage) been tested using cyclic loading.

Another major factor influencing the behavior of shear walls is the aspect ratio of the wall. The *Uniform Building Code (UBC)* (ICBO, 1994a) limits the aspect ratio (height-to-width) to 3.5:1. After the 1994 Northridge earthquake, the city of Los Angeles reduced the allowable aspect ratio to 2:1, pending additional tests. The interaction of the floor and roof with the wall, the end conditions of the wall, and the redundancy or number of walls along any wall line would affect the wall behavior for walls with the same aspect ratio. In addition, the rigidity of the tie-downs at the wall ends has an important effect in the behavior of narrow walls.

Dissimilar wall sheathing materials on opposite sides of a wall should not be combined when calculating the capacity of the wall. Similarly, different walls sheathed with dissimilar materials along the same line of lateral force resistance should be analyzed based on only one type of sheathing. The wall sheathing with the greatest capacity should be used for determining capacity. The walls should also be analyzed based on the relative rigidity and capacity of the materials to determine if performance of the "nonparticipating" material will be acceptable.

For uplift calculations on shear-wall elements, the overturning moment on the wall should be based on the calculated load on the wall from the base shear and the use of an appropriate *m* factor for the uplift connector. As an alternative, the uplift can be based on a lateral load equal to 1.2 times the yield capacity of the wall. However, no *m* factor is involved in the demand versus capacity equation, and the uplift connector yield capacity should not be exceeded.

Connections between elements, drag ties, struts, and other structured members should be based on the calculated load to the connection under study, and analyzed. As an alternative, the connection can be analyzed for a maximum load, at the connection, of 1.2 times the yield capacity of the weaker element. The yield capacity of the connection should not be exceeded and no *m* factor is used in the analysis.

For wood and light frame shear walls, the important limit states are sheathing failure, connection failure, tie-down failure, and excessive deflection. Limit states define the point of life safety and, often, of structural stability. To reduce damage or retain usability immediately after an earthquake, deflection must be limited (see Section 2.5). The ultimate capacity is the maximum capacity the assembly can resist, regardless of the deflection. See Section 8.5.11 for the effect of openings in diaphragms. The expected capacity, Q_{CE}, is equal to the yield capacity of the shear wall, V_y.

8.4.1 Types of Light Frame Shear Walls

8.4.1.1 Existing Shear Walls

A. Single Layer Horizontal Lumber Sheathing or Siding

Typically, 1" x horizontal sheathing or siding is applied directly to studs. Forces are resisted by nail couples. Horizontal boards, from 1" x 4" to 1" x 12" typically are nailed to 2" x or greater width studs with two or more nails (typically 8d or 10d) per stud. The strength and stiffness degrade with cyclic loading. (See Section 3.3.1.3 and Table 3-1 for the appropriate C_2 value.)

B. Diagonal Lumber Sheathing

Typically, 1" x 6" or 1" x 8" diagonal sheathing, applied directly to the studs, resists lateral forces primarily by triangulation (i.e., direct tension and compression). Sheathing boards are installed at a 45-degree angle to studs, with three or more nails (typically 8d) per stud, and to sill and top plates. A second layer of diagonal sheathing is sometimes added on top of the first layer, at 90 degrees to the first layer (called Double Diagonal Sheathing), for increased load capacity and stiffness.

C. Vertical Wood Siding Only

Typically, 1" x 8", 1" x 10", or 1" x 12" vertical boards are nailed directly to 2" x or greater width studs and blocking with 8d to 10d galvanized nails. The lateral forces are resisted by nail couples, similarly to horizontal siding. The strength and stiffness degrade with cyclic loading. (See Section 3.3.1.3 and Table 3-1 for the appropriate C_2 value.)

D. Wood Siding over Horizontal Sheathing

Typically, siding is nailed with 8d to 10d galvanized nails through the sheathing to the studs. Lateral forces are resisted by nail couples for both layers. The strength and stiffness degrade with cyclic loading. (See Section 3.3.1.3 and Table 3-1 for the appropriate C_2 value.)

E. Wood Siding over Diagonal Sheathing

Typically, siding is nailed with 8d or 10d galvanized nails to and through the sheathing into the studs. Diagonal sheathing provides most of the lateral resistance by truss-shear action.

F. Structural Panel or Plywood Panel Sheathing or Siding

Typically, 4' x 8' panels are applied vertically or horizontally to 2" x or greater studs and nailed with 6d to 10d nails. These panels resist lateral forces by panel diaphragm action.

G. Stucco on Studs (over sheathing or wire backed building paper)

Typically, 7/8-inch portland cement plaster is applied on wire lath or expanded metal lath. Wire lath or expanded metal lath is nailed to the studs with 11 gage nails or 16 gage staples at 6 inches on center. This assembly resists lateral forces by panel diaphragm action. The strength and stiffness degrade with cyclic loading. (See Section 3.3.1.3 and Table 3-1 for the appropriate C_2 value.)

H. Gypsum Plaster on Wood Lath

Typically, 1-inch gypsum plaster is keyed onto spaced 1-1/4-inch wood lath that is nailed to studs with 13 gage nails. Gypsum plaster on wood lath resists lateral forces by panel diaphragm-shear action. The strength and stiffness degrade with cyclic loading. (See Section 3.3.1.3 and Table 3-1 for the appropriate C_2 value.)

I. Gypsum Plaster on Gypsum Lath

Typically, 1/2-inch plaster is glued or keyed to 16-inch x 48-inch gypsum lath, which is nailed to studs with 13 gage nails. Gypsum plaster on gypsum lath resists lateral loads by panel diaphragm action. The strength and stiffness degrade with cyclic loading. (See Section 3.3.1.3 and Table 3-1 for the appropriate C_2 value.)

J. Gypsum Wallboard or Drywall

Typically, 4' x 8' to 4' x 12' panels are laid-up horizontally or vertically and nailed to studs or blocking with 5d to 8d cooler nails at 4 to 7 inches on center. Multiple layers are utilized in some situations. The assembly resists lateral forces by panel diaphragm-shear action. The strength and stiffness degrade with cyclic loading. (See Section 3.3.1.3 and Table 3-1 for the appropriate C_2 value.)

K. Gypsum Sheathing

Typically, 4' x 8' to 4' x 12' panels are laid-up horizontally or vertically and nailed to studs or blocking with galvanized 11 gage 7/16-inch diameter head nails at 4 to 7 inches on center. Gypsum sheathing is usually installed on the exterior of structures with siding over it in order to improve fire resistance. Lateral forces are resisted by panel diaphragm action. The strength and stiffness degrades with cyclic loading. (See Section 3.3.1.3 and Table 3-1 for the appropriate C_2 value.)

L. Plaster on Metal Lath

Typically, 1-inch gypsum plaster is applied on expanded wire lath that is nailed to the studs. Lateral forces are resisted by panel diaphragm action. The strength and stiffness degrade with cyclic loading. (See Section 3.3.1.3 and Table 3-1 for the appropriate C_2 value.)

M. Horizontal Lumber Sheathing with Cut-In Braces or Diagonal Blocking

This is installed in the same manner as horizontal sheathing, except the wall is braced at the corners with cut-in (or let-in) braces or blocking. The bracing is usually installed at a 45-degree angle and nailed with 8d or 10d nails at each stud, and at the top and bottom plates. Bracing provides only nominal increase in resistance. The strength and stiffness degrade with cyclic loading. (See Section 3.3.1.3 and Table 3-1 for the appropriate C_2 value.)

N. Fiberboard or Particleboard Sheathing

Typically, 4' x 8' panels are applied directly to the studs with nails. The fiberboard requires nails (typically 8d) with large heads such as roofing nails. Lateral loads are resisted by panel diaphragm action. The strength and stiffness degrade with cyclic loading. (See Section 3.3.1.3 and Table 3-1 for the appropriate C_2 value.)

8.4.1.2 Shear Wall Enhancements for Rehabilitation

A. Structural Panel Sheathing Added to Unfinished Stud Walls

Wall shear capacity and stiffness can be increased by adding structural panel sheathing to one side of unfinished stud walls, such as cripple walls or attic end walls.

B. Structural Panel Sheathing Overlay of Existing Shear Walls

For a moderate increase in shear capacity and stiffness that can be applied in most places in most structures, the existing wall covering can be overlaid with structural panel sheathing; for example, plywood sheathing can be applied over an interior wall finish. For exterior applications, the structural panel can be placed over the exterior finish and nailed directly through it to the studs. This rehabilitation procedure typically can be used for the following shear walls, which are described in Section 8.4.1.1:

- Single layer horizontal lumber sheathing or siding
- Single layer diagonal lumber sheathing
- Vertical wood siding only
- Gypsum plaster or wallboard on studs (also on gypsum lath and gypsum wallboard)
- Gypsum sheathing
- Horizontal lumber sheathing with cut-in braces or diagonal blocking
- Fiberboard or particleboard sheathing

The enhanced shear wall is evaluated in accordance with Section 8.4.9, discounting the original sheathing and reducing the yield capacity of the overlay material by 20%.

C. Structural Panel Sheathing Added Under Existing Wall Covering

To obtain a significant increase in shear capacity, the existing wall covering can be removed; structural panel sheathing, connections, and tie-downs added; and the wall covering replaced. In some cases, where earthquake loads are large, this may be the best method of rehabilitation. This rehabilitation procedure can be used on any of the existing shear wall assemblies. Additional framing members can be added if necessary, and the structural panels can be cut to fit existing stud spacings.

D. Increased Attachment

For existing structural panel sheathed walls, additional nailing will result in higher capacity and increased stiffness. Other connectors—such as collector straps, splice straps, or tie-downs—are often necessary to increase the rigidity and capacity of existing structural panel shear walls. Increased ductility will not necessarily result from the additional nailing. Access to these shear walls will often require the removal and replacement of existing finishes.

E. Rehabilitation of Connections

Most shear wall rehabilitation procedures require a check of all existing connections, especially to diaphragms and foundations. Additional blocking between floor or roof joists at shear walls is often needed on existing structures. The blocking must be connected to the shear wall and the diaphragm to provide a load path for lateral loads. Sheet metal framing clips can be used to provide a verifiable connection between the wall framing, the blocking, and the diaphragm. Framing clips are also often used for connecting blocking or rim joists to sill plates.

The framing in existing buildings is usually very dry, hard, and easily split. Care must be taken not to split the existing framing when adding connectors. Predrilling holes for nails will reduce splitting, and framing clips that use small nails are less likely to split the existing framing.

When existing shear walls are overlaid with structural panels, the connections of the structural panels to the existing framing must be considered. Splitting can occur in both the wood sheathing and the framing. The length of nails needed to achieve full capacity attachment in the existing framing must be determined. This length will vary with the thickness of the existing wall covering. Sometimes staples are used instead of nails to prevent splitting. The overlay is stapled to the wood sheathing instead of the framing. Nails are recommended for overlay attachment to the underlying framing. In some cases, new blocking at structural panel joints may also be needed.

When framing members or blocking are added to a structure, the wood should be kiln-dried or well-seasoned to prevent it from shrinking away from the existing framing or splitting.

8.4.1.3 New Shear Walls Sheathed with Structural Panels or Plywood Panel Sheathing or Siding

New shear walls using the existing framing or new framing are sheathed with structural panels (i.e., plywood or oriented strand board). The thickness and grade of these panels can vary. In most cases, the panels are placed vertically and fastened directly to the studs and plates. This reduces the need for blocking at the joints. All edges of panels must be blocked to obtain full capacity. The thickness, size, and number of fasteners, and aspect ratio and connections will determine the capacity of the new walls. Additional information on the various panels available and their application can be found in documents from the American Plywood Association (APA), such as APA (1983).

8.4.2 Light Gage Metal Frame Shear Walls

8.4.2.1 Existing Light Gage Metal Frame Shear Walls

A. Plaster on Metal Lath

Typically, 1 inch of gypsum plaster is applied to metal lath or expanded metal that is connected to the metal framing with wire ties.

B. Gypsum Wallboard

Typically, 4' x 8' to 4' x 12' panels are laid-up horizontally and screwed with No. 6 x 1-inch-long self-tapping screws to studs at 4 to 7 inches on center.

C. Plywood or Structural Panels

Typically, the structural panels are applied vertically and screwed to the studs and track with No. 8 to No. 12 self-tapping screws.

8.4.2.2 Light Gage Metal Frame Enhancements for Rehabilitation

A. Addition of Plywood Structural Panels to Existing Metal Stud Walls

Any existing covering other than plywood is removed and replaced with structural panels. Connections to the diaphragm(s) and the foundation must be checked and may need to be strengthened.

B. Existing Plywood or Structural Panels on Metal Studs

Added screws and possibly additional connections to diaphragms and foundation may be required.

8.4.2.3 New Light Gage Metal Frame Shear Walls

A. Plywood or Structural Panels

Refer to Section 8.4.1.3.

8.4.3 Knee-Braced and Miscellaneous Timber Frames

8.4.3.1 Knee-Braced Frames

Knee-braced frames produce moment-resisting joints by the addition of diagonal members between columns and beams. The resulting “semi-rigid” frame resists lateral loads. The moment-resisting capacity of knee-braced frames varies widely. The controlling part of the assembly is usually the connection; however, bending of members can be the controlling feature of some frames. Once the capacity of the connection is determined, members can be checked and the capacity of the frame can be determined by statics. For a detailed discussion on connections, see Section C8.3.2.2B in the *Commentary*.

8.4.3.2 Rod-Braced Frames

Similarly to knee-braced frames, the connections of rods to timber framing will usually govern the capacity of the rod-braced frame. Typically, the rods act only in tension. Once the capacity of the connection is determined, the capacity of the frame can be determined by statics. See Section 8.3.2.2B.

8.4.4 Single Layer Horizontal Lumber Sheathing or Siding Shear Walls

8.4.4.1 Stiffness for Analysis

Horizontal lumber sheathed shear walls are weak and very flexible and have long periods of vibration. These shear walls are suitable only where earthquake shear loads are low and deflection control is not required. The deflection of these shear walls can be approximated by Equation 8-1:

$$\Delta_y = v_y h/G_d + (h/b)d_a \tag{8-1}$$

where:

b = Shear wall length, ft
h = Shear wall height, ft
v_y = Shear at yield, lb/ft
G_d = Shear stiffness in lb/in.
Δ_y = Calculated shear wall deflection at yield, in.
d_a = Elongation of anchorage at end of wall determined by anchorage details and load magnitude, in.

For horizontal lumber sheathed shear walls, G_d = 2,000 lb/in.

8.4.4.2 Strength Acceptance Criteria

Horizontal sheathing or siding has an estimated yield capacity of 80 pounds per linear foot. This capacity is dependent on the width of the boards, spacing of the studs, and the size, number, and spacing of the nails. Allowable capacities are listed for various configurations, together with a description of the nail couple method, in the *Western Woods Use Book* (WWPA, 1983). See also ATC (1981) for a discussion of the nail couple.

8.4.4.3 Deformation Acceptance Criteria

The deformation acceptance criteria are determined by the capacity of lateral- and gravity-load-resisting components and elements to deform with limited damage or without failure. Excessive deflection could result in major damage to the structure and/or its contents. See Table 8-1 for *m* factors for use in the LSP in performing design analyses.

The coordinates for the normalized force-deflection curve used for modeling in connection with the nonlinear procedures (Figure 8-1) are shown in Table 8-2. The values in this table refer to Figure 8-1 in the following way. Distance *d* is considered the maximum deflection at the point of first loss of strength. Distance *e* is the maximum deflection at a strength or capacity equal to value *c*. Figure 8-1 also shows the deformation ratios for IO, LS, and CP Performance Levels for primary components. (See Chapter 3 for the use of the force-deflection curve in the NSP.)

Deformation acceptance criteria for use in connection with nonlinear procedures are given in footnotes of Table 8-2 for primary and secondary components, respectively.

Table 8-1 Numerical Acceptance Factors for Linear Procedures—Wood Components

		***m* Factors for Linear Procedures**[2]				
		Primary			**Secondary**	
		IO	**LS**	**CP**	**LS**	**CP**
Shear Walls	**Height/Length Ratio (*h*/*L*)**[1]					
Horizontal 1" x 6" Sheathing	$h/L < 1.0$	1.8	4.2	5.0	5.0	5.5
Horizontal 1" x 10" Sheathing	$h/L < 1.0$	1.6	3.4	4.0	4.0	5.0
Horizontal Wood Siding Over Horizontal 1" x 6" Sheathing	$h/L < 1.5$	1.4	2.6	3.0	3.1	4.0
Horizontal Wood Siding Over Horizontal 1" x 10" Sheathing	$h/L < 1.5$	1.3	2.3	2.6	2.8	3.0
Diagonal 1" x 6" Sheathing	$h/L < 1.5$	1.5	2.9	3.3	3.4	3.8
Diagonal 1" x 8" Sheathing	$h/L < 1.5$	1.4	2.7	3.1	3.1	3.6
Horizontal Wood Siding Over Diagonal 1" x 6" Sheathing	$h/L < 2.0$	1.3	2.2	2.5	2.5	3.0
Horizontal Wood Siding Over Diagonal 1" x 8" Sheathing	$h/L < 2.0$	1.3	2.0	2.3	2.5	2.8
Double Diagonal 1" x 6" Sheathing	$h/L < 2.0$	1.2	1.8	2.0	2.3	2.5
Double Diagonal 1" x 8" Sheathing	$h/L < 2.0$	1.2	1.7	1.9	2.0	2.5
Vertical 1" x 10" Sheathing	$h/L < 1.0$	1.5	3.1	3.6	3.6	4.1

1. For ratios greater than the maximum listed values, the component is considered not effective in resisting lateral loads.

2 Linear interpolation is permitted for intermediate value if h/L has asterisks.

Table 8-1 Numerical Acceptance Factors for Linear Procedures—Wood Components (continued)

		m Factors for Linear Procedures[2]				
		Primary			Secondary	
		IO	LS	CP	LS	CP
Structural Panel or Plywood Panel Sheathing or Siding	*h*/*L* < 1.0*	1.7	3.8	4.5	4.5	5.5
	h/*L* > 2.0* *h*/*L* < 3.5	1.4	2.6	3.0	3.0	4.0
Stucco on Studs	*h*/*L*<1.0 *	1.5	3.1	3.6	3.6	4.0
	h/*L* = 2.0 *	1.3	2.2	2.5	2.5	3.0
Stucco over 1" x Horizontal Sheathing	*h*/*L* < 2.0	1.5	3.0	3.5	3.5	4.0
Gypsum Plaster on Wood Lath	*h*/*L* < 2.0	1.7	3.9	4.6	4.6	5.1
Gypsum Plaster on Gypsum Lath	*h*/*L* < 2.0	1.8	4.2	5.0	4.2	5.5
Gypsum Plaster on Metal Lath	*h*/*L* < 2.0	1.7	3.7	4.4	3.7	5.0
Gypsum Sheathing	*h*/*L* < 2.0	1.9	4.7	5.7	4.7	6.0
Gypsum Wallboard	*h*/*L* < 1.0 *	1.9	4.7	5.7	4.7	6.0
	h/*L* = 2.0 *	1.6	3.4	4.0	3.8	4.5
Horizontal 1" x 6" Sheathing With Cut-In Braces or Diagonal Blocking	*h*/*L* < 1.0	1.7	3.7	4.4	4.2	4.8
Fiberboard or Particleboard Sheathing	*h*/*L* < 1.5	1.6	3.2	3.8	3.8	5.0
Diaphragms	**Length/Width Ratio (*L*/*b*)[1]**					
Single Straight Sheathing, Chorded	*L*/*b* < 2.0	1	2.0	2.5	2.4	3.1
Single Straight Sheathing, Unchorded	*L*/*b* < 2.0	1	1.5	2.0	1.8	2.5
Double Straight Sheathing, Chorded	*L*/*b* < 2.5	1.25	2.0	2.5	2.3	2.8
Double Straight Sheathing, Unchorded	*L*/*b* < 2.5	1	1.5	2.0	1.8	2.3
Single Diagonal Sheathing, Chorded	*L*/*b* < 2.5	1.25	2.0	2.5	2.3	2.9
Single Diagonal Sheathing, Unchorded	*L*/*b* < 2.0	1	1.5	2.0	1.8	2.5
Straight Sheathing Over Diagonal Sheathing, Chorded	*L*/*b* < 3.0	1.5	2.5	3.0	2.8	3.5
Straight Sheathing Over Diagonal Sheathing, Unchorded	*L*/*b* < 2.5	1.25	2.0	2.5	2.3	3.0
Double Diagonal Sheathing, Chorded	*L*/*b* < 3.5	1.5	2.5	3.0	2.9	3.5
Double Diagonal Sheathing, Unchorded	*L*/*b* < 3.5	125	2.0	2.5	2.4	3.1
Wood Structural Panel, Blocked, Chorded	*L*/*b* < 3.0 * *L*/*b* = 4*	1.5 1.5	3.0 2.5	4.0 3.0	3.5 2.8	4.5 3.5
Wood Structural Panel, Unblocked, Chorded	*L*/*b* < 3* *L*/*b* = 4*	1.5 1.5	2.5 2.0	3.0 2.5	2.9 2.6	4.0 3.2
Wood Structural Panel, Blocked, Unchorded	*L*/*b* < 2.5 *L*/*b* = 3.5	1.25 1.25	2.5 2.0	3.0 2.5	2.9 2.6	4.0 3.2
Wood Structural Panel, Unblocked, Unchorded	*L*/*b* < 2.5 *L*/*b* = 3.5	1.25 1.0	2.0 1.5	2.5 2.0	2.4 2.0	3.0 2.6
Wood Structural Panel Overlay on Sheathing, Chorded	*L*/*b* < 3* *L*/*b* = 4*	1.5 1.5	2.5 2.0	3.0 2.5	2.9 2.6	4.0 3.2
Wood Structural Panel Overlay on Sheathing, Unchorded	*L*/*b* < 2.5 *L*/*b* = 3.5	1.25 1.0	2.0 1.5	2.5 2.0	2.4 1.9	3.0 2.6

1. For ratios greater than the maximum listed values, the component is considered not effective in resisting lateral loads.
2 Linear interpolation is permitted for intermediate value if h/L has asterisks.

Table 8-1 Numerical Acceptance Factors for Linear Procedures—Wood Components (continued)

	m Factors for Linear Procedures[2]				
	Primary			Secondary	
	IO	LS	CP	LS	CP
Component/Element					
Frame elements subject to axial and bending stresses	1.0	2.5	3.0	2.5	4.0
Connections					
Nails - 8d and larger - Wood to Wood	2.0	6.0	8.0	8.0	9.0
Nails - 8d and larger - Metal to Wood	2.0	4.0	6.0	5.0	7.0
Screws - Wood to Wood	1.2	2.0	2.2	2.0	2.5
Screws - Metal to Wood	1.1	1.8	2.0	1.8	2.3
Lag Bolts - Wood to Wood	1.4	2.5	3.0	2.5	3.3
Lag Bolts - Metal to Wood	1.3	2.3	2.5	2.4	3.0
Machine Bolts - Wood to Wood	1.3	3.0	3.5	3.3	3.9
Machine Bolts - Metal to Wood	1.4	2.8	3.3	3.1	3.7
Split Rings and Shear Plates	1.3	2.2	2.5	2.3	2.7
Bolts - Wood to Concrete or Masonry	1.4	2.7	3.0	2.8	3.5

1. For ratios greater than the maximum listed values, the component is considered not effective in resisting lateral loads.
2 Linear interpolation is permitted for intermediate value if h/L has asterisks.

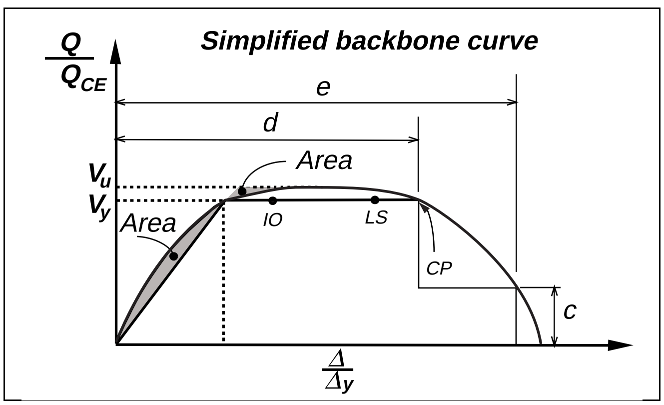

Figure 8-1 Normalized Force versus Deformation Ratio for Wood Elements

8.4.4.4 Connections

The connections between parts of the shear wall assembly and other elements of the lateral-force-resisting system must be investigated and analyzed. The capacity and ductility of these connections will often determine the failure mode as well as the capacity of the assembly. Ductile connections with sufficient capacity will give acceptable and expected performance (see Section 8.3.2.2B).

8.4.5 Diagonal Lumber Sheathing Shear Walls

8.4.5.1 Stiffness for Analysis

Diagonal lumber sheathed shear walls are stiffer and stronger than horizontal sheathed shear walls. They also provide greater stiffness for deflection control, and thereby greater damage control. The deflection of these shear walls can be determined using Equation 8-1, with G_d = 8,000 lb/in. for single layer diagonal siding and G_d = 18,000 lb/in. for double diagonal siding.

Table 8-2 Normalized Force-Deflection Curve Coordinates for Nonlinear Procedures—Wood Components

		d	e	c
Shear Wall Type - Types of Existing Wood and Light Frame Shear Walls	**Height/Length Ratio h/L[1]**			
Horizontal 1" x 6" Sheathing	$h/L < 1.0$	5.0	6.0	0.3
Horizontal 1" x 10" Sheathing	$h/L < 1.0$	4.0	5.0	0.3
Horizontal Wood Siding Over Horizontal 1" x 6" Sheathing	$h/L < 1.5$	3.0	4.0	0.2
Horizontal Wood Siding Over Horizontal 1" x 10" Sheathing	$h/L < 1.5$	2.6	3.6	0.2
Diagonal 1" x 6" Sheathing	$h/L < 1.5$	3.3	4.0	0.2
Diagonal 1" x 8" Sheathing	$h/L < 1.5$	3.1	4.0	0.2
Horizontal Wood Siding Over Diagonal 1" x 6" Sheathing	$h/L < 2.0$	2.5	3.0	0.2
Horizontal Wood Siding Over Diagonal 1" x 8" Sheathing	$h/L < 2.0$	2.3	3.0	0.2
Double Diagonal 1" x 6" Sheathing	$h/L < 2.0$	2.0	2.5	0.2
Double Diagonal 1" x 8" Sheathing	$h/L < 2.0$	2.0	2.5	0.2
Vertical 1" x 10" Sheathing	$h/L < 1.0$	3.6	4.0	0.3
Structural Panel or Plywood Panel Sheathing or Siding	$h/L < 1.0$*	4.5	5.5	0.3
	$h/L > 2.0$* $h/L < 3.5$	3.0	4.0	0.2
Stucco on Studs	$h/L < 1.0$*	3.6	4.0	0.2
	$h/L = 2.0$*	2.5	3.0	0.2
Stucco over 1" x Horizontal Sheathing	$h/L < 2.0$	3.5	4.0	0.2
Gypsum Plaster on Wood Lath	$h/L < 2.0$	4.6	5.0	0.2
Gypsum Plaster on Gypsum Lath	$h/L < 2.0$	5.0	6.0	0.2
Gypsum Plaster on Metal Lath	$h/L < 2.0$	4.4	5.0	0.2
Gypsum Sheathing	$h/L < 2.0$	5.7	6.3	0.2
Gypsum Wallboard	$h/L < 1.0$*	5.7	6.3	0.2
	$h/L = 2.0$*	4.0	5.0	0.2
Horizontal 1" x 6" Sheathing With Cut-In Braces or Diagonal Blocking	$h/L < 1.0$	4.4	5.0	0.2
Fiberboard or Particleboard Sheathing	$h/L < 1.5$	3.8	4.0	0.2
Diaphragm Type - Horizontal Wood Diaphragms	**Length/Width Ratio (L/b)[1]**			
Single Straight Sheathing, Chorded	$L/b < 2.0$	2.5	3.5	0.2
Single Straight Sheathing, Unchorded	$L/b < 2.0$	2.0	3.0	0.3
Double Straight Sheathing, Chorded	$L/b < 2.0$	2.5	3.5	0.2

1. For ratios greater than the maximum listed values, the component is considered not effective in resisting lateral loads.

Notes: (a) Acceptance criteria for primary components

(Δ/Δ_y) IO = 1.0 + 0.2 (d - 1.0)
(Δ/Δ_y) LS = 1.0 + 0.8 (d - 1.0)
(Δ/Δ_y) CP = d

(b) Acceptance criteria for secondary components

(Δ/Δ_y) LS = d
(Δ/Δ_y) CP = e

(c) Linear interpolation is permitted for intermediate values if h/L or L/b has asterisks.

Table 8-2 Normalized Force-Deflection Curve Coordinates for Nonlinear Procedures—Wood Components (continued)

		d	e	c
Double Straight Sheathing, Unchorded	*L*/*b* < 2.0	2.0	3.0	0.3
Single Diagonal Sheathing, Chorded	*L*/*b* < 2.0	2.5	3.5	0.2
Single Diagonal Sheathing, Unchorded	*L*/*b* < 2.0	2.0	3.0	0.3
Straight Sheathing Over Diagonal Sheathing, Chorded	*L*/*b* < 2.0	3.0	4.0	0.2
Straight Sheathing Over Diagonal Sheathing, Unchorded	*L*/*b* < 2.0	2.5	3.5	0.3
Double Diagonal Sheathing, Chorded	*L*/*b* < 2.0	3.0	4.0	0.2
Double Diagonal Sheathing, Unchorded	*L*/*b* < 2.0	2.5	3.5	0.2
Wood Structural Panel, Blocked, Chorded	*L*/*b* < 3* *L*/*b* = 4*	4.0 3.0	5.0 4.0	0.3
Wood Structural Panel, Unblocked, Chorded	*L*/*b* < 3* *L*/*b* = 4*	3.0 2.5	4.0 3.5	0.3
Wood Structural Panel, Blocked, Unchorded	*L*/*b* < 2.5* *L*/*b* = 3.5*	3.0 2.5	4.0 3.5	0.3
Wood Structural Panel, Unblocked, Unchorded	*L*/*b* < 2.5* *L*/*b* = 3.5*	2.5 2.0	3.5 3.0	0.4
Wood Structural Panel Overlay On Sheathing, Chorded	*L*/*b* < 3* *L*/*b* = 4*	3.0 2.5	4.0 3.5	0.3
Wood Structural Panel Overlay On Sheathing, Unchorded	*L*/*b* < 2.5* *L*/*b* = 3.5*	2.5 2.0	3.5 3.0	0.4
Connection Type				
Nails - Wood to Wood		7.0	8.0	0.2
Nails - Metal to Wood		5.5	7.0	0.2
Screws - Wood to Wood		2.5	3.0	0.2
Screws - Wood to Metal		2.3	2.8	0.2
Lag Bolts - Wood to Wood		2.8	3.2	0.2
Lag Bolts - Metal to Wood		2.5	3.0	0.2
Bolts - Wood to Wood		3.0	3.5	0.2
Bolts - Metal to Wood		2.8	3.3	0.2

1. For ratios greater than the maximum listed values, the component is considered not effective in resisting lateral loads.

Notes: (a) Acceptance criteria for primary components

(Δ/Δ_y) IO = 1.0 + 0.2 (d - 1.0)
(Δ/Δ_y) LS = 1.0 + 0.8 (d - 1.0)
(Δ/Δ_y) CP = d

(b) Acceptance criteria for secondary components

(Δ/Δ_y) LS = d
(Δ/Δ_y) CP = e

(c) Linear interpolation is permitted for intermediate values if h/L or L/b has asterisks.

8.4.5.2 Strength Acceptance Criteria

Diagonal sheathing has an estimated yield capacity of approximately 700 pounds per linear foot for single layer and 1300 pounds per linear foot for double diagonal sheathing. This capacity is dependent on the width of the boards, the spacing of the studs, the size of nails, the number of nails per board, and the boundary conditions. Allowable capacities are listed for various configurations in WWPA (1983).

8.4.5.3 Deformation Acceptance Criteria

The deformation acceptance criteria will be determined by the capacity of lateral- and gravity-load-resisting elements to deform without failure. See Table 8-1 for *m* factors for use in the LSP.

The coordinates for the normalized force-deflection curve used in the nonlinear procedures are shown in Table 8-2. The values in this table refer to Figure 8-1 in the following way. Distance *d* is considered the maximum deflection at the point of loss of strength. Distance *e* is the maximum deflection at a strength or capacity equal to value *c*. (See Chapter 3 for the use of the force deflection curve in the NSP.)

8.4.5.4 Connections

See Sections 8.3.2.2B and 8.4.4.4.

8.4.6 Vertical Wood Siding Shear Walls

8.4.6.1 Stiffness for Analysis

Vertical wood siding has a very low lateral-force-resistance capacity and is very flexible. These shear walls are suitable only where earthquake shear loads are very low and deflection control is not needed. The deflection of these shear walls can be determined using Equation 8-1, with G_d = 1,000 lb/in.

8.4.6.2 Strength Acceptance Criteria

Vertical siding has a yield capacity of approximately 70 pounds per linear foot. This capacity is dependent on the width of the boards, the spacing of the studs, the spacing of blocking, and the size, number, and spacing of the nails. The nail couple method can be used to calculate the capacity of vertical wood siding, in a manner similar to the method used for horizontal siding.

8.4.6.3 Deformation Acceptance Criteria

See Section 8.4.5.3.

8.4.6.4 Connections

The load capacity of the vertical siding is low; this makes the capacity of connections between the shear wall and the other elements of secondary concern (see Section 8.3.2.2B).

8.4.7 Wood Siding over Horizontal Sheathing Shear Walls

8.4.7.1 Stiffness for Analysis

Double layer horizontal sheathed shear walls are stiffer and stronger than single layer horizontal sheathed shear walls. These shear walls are often suitable for resisting earthquake shear loads that are low to moderate in magnitude. They also provide greater stiffness for deflection control, and thereby greater damage control. The deflection of these shear walls can be determined using Equation 8-1, with G_d = 4,000 lb/in.

8.4.7.2 Strength Acceptance Criteria

Wood siding over horizontal sheathing has a yield capacity of approximately 500 pounds per linear foot. This capacity is dependent on the width of the boards, the spacing of the studs, the size, number, and spacing of the nails, and the location of joints.

8.4.7.3 Deformation Acceptance Criteria

See Section 8.4.5.3.

8.4.7.4 Connections

See Sections 8.3.2.2B and 8.4.4.4.

8.4.8 Wood Siding over Diagonal Sheathing Shear Walls

8.4.8.1 Stiffness for Analysis

Horizontal wood siding over diagonal sheathing will provide stiff, strong shear walls. These shear walls are often suitable for resisting earthquake shear loads that are moderate in magnitude. They also provide good stiffness for deflection control and damage control. The deflection of these shear walls can be approximated by using Equation 8-1, with G_d = 11,000 lb/in.

8.4.8.2 Strength Acceptance Criteria

Wood siding over diagonal sheathing has an estimated yield capacity of approximately 1,100 pounds per linear foot. This capacity is dependent on the width of the boards, the spacing of the studs, the size, number, and spacing of the nails, the location of joints, and the boundary conditions.

8.4.8.3 Deformation Acceptance Criteria

See Section 8.4.5.3.

8.4.8.4 Connections

See Sections 8.3.2.2B and 8.4.4.4.

8.4.9 Structural Panel or Plywood Panel Sheathing Shear Walls

8.4.9.1 Stiffness for Analysis

The response of wood structural shear walls is dependent on the thickness of the wood structural panels, the height-to-length (h/L) ratio, the nailing pattern, and other factors. The approximate deflection of wood structural shear walls at yield can be determined using Equation 8-2:

$$\Delta_y = 8\, v_y\, h^3/(E\, A\, b) + v_y\, h/(G\, t) + 0.75h\, e_n + (h/b)d_a \qquad (8\text{-}2)$$

where:

v_y	=	Shear at yield in the direction under consideration in lb/ft
h	=	Wall height, ft
E	=	Modulus of wood end boundary member, psi
A	=	Area of boundary member cross section, in.^{2}
b	=	Wall width, ft
G	=	Modulus of rigidity of plywood, psi
t	=	Effective thickness of structural panel, in.
d_a	=	Deflection at yield of tie-down anchorage or deflection at load level to anchorage at end of wall, anchorage details, and dead load, in.
e_n	=	Nail deformation, in.

For 6d nails at yield: $e_n = .10$

For 8d nails at yield: $e_n = .06$

For 10d nails at yield: $e_n = .04$

8.4.9.2 Strength Acceptance Criteria

Shear capacities of wood structural panel shear walls are primarily dependent on the nailing at the plywood panel edges, and the thickness and grade of the plywood. The yield shear capacity, V_y, of wood structural shear walls can be calculated as follows:

$$V_y = .8V_u \qquad (8\text{-}3)$$

Values of ultimate capacity , V_u, of structural panel shear walls are provided in Table 8-3.

If there is no ultimate load for the assembly, use:

$$Q_{CE} = V_u = 6.3Zs/a \qquad (8\text{-}4)$$

where:

Z	=	Nail value from NDS (1991)
s	=	Minimum $[m\text{-}1 \text{ or } (n\text{-}1)(a/h)]$
m	=	Number of nails along the bottom of one panel
n	=	Number of nails along one side of one panel
a	=	Length of one panel
h	=	Height of one panel

8.4.9.3 Deformation Acceptance Criteria

See Section 8.4.5.3.

8.4.9.4 Connections

See Sections 8.3.2.2B and 8.4.4.4.

8.4.10 Stucco on Studs, Sheathing, or Fiberboard Shear Walls

8.4.10.1 Stiffness for Analysis

Stucco is brittle and the lateral-force-resistance capacity of stucco shear walls is low. However, the walls are stiff until cracking occurs. These shear walls are suitable only where earthquake shear loads are low. The deflection of these shear walls can be determined using Equation 8-1 with G_d = 14,000 lb/in.

8.4.10.2 Strength Acceptance Criteria

Stucco has a yield capacity of approximately 350 pounds per linear foot. This capacity is dependent on the attachment of the stucco netting to the studs and the embedment of the netting in the stucco.

8.4.10.3 Deformation Acceptance Criteria

See Section 8.4.5.3.

8.4.10.4 Connections

The connection between the stucco netting and the framing is of primary concern. Of secondary concern is the connection of the stucco to the netting. Unlike plywood, the tensile capacity of the stucco material

Table 8-3 Ultimate Capacities of Structural Panel Shear Walls[2, 3, 5, 6]

Panel Grade	Minimum Nominal Panel Thickness (inches)	Minimum Nail Penetration in Framing[4] (inches)	Nail Size[4] (Common or Galvanized Box)	Nail Spacing at Panel Edges (in.) Ultimate Capacities (lb/ft)			
				6"	4"	3"[1]	2"[1]
Structural 1	5/16	1 1/4	6d	700	1010	1130	1200
	3/8	1 1/2	8d	750	1080	1220	1540
	7/16			815	1220	1340	1590
	15/32			880	1380	1550	1620
	15/32	1 5/8	10d[1]	1130	1500	1700	2000
C-D, C-C	5/16	1 1/4	6d	650	700	900	1200
Sheathing, plywood panel siding (and other grades covered in UBC Standard 23-2 or 23-3), structural particleboard	3/8			680	800	1000	1350
	3/8	1 1/2	8d	700	880	1200	1500
	7/16			720	900	1300	1560
	15/32			820	1040	1420	1600
	15/32	1 5/8	10d[1]	900	1400	1500	1900
	19/32			1000	1500	1620	1950

1. 3x or greater framing at plywood joints.
2. Panels applied directly to framing, blocked at all edges.
3. Value extrapolated from cyclic testing.
4. For other nail sizes or nail penetration less than indicated, adjust values based on calculated nail strength (see AF&PA, 1991).
5. Values are for panels on one side. Values may be doubled for panels on both sides.
6. Use 80% of values listed for yield capacity.

(portland cement) rather than the connections, will often govern failure. The connections between the shear wall and foundation and between the shear wall and diaphragm must be investigated. See Section 8.3.2.2B.

8.4.11 Gypsum Plaster on Wood Lath Shear Walls

8.4.11.1 Stiffness for Analysis

Gypsum plaster shear walls are similar to stucco, except their strength is lower. Again, the walls are stiff until failure. These shear walls are suitable only where earthquake shear loads are very low. The deflection of these shear walls can be determined using Equation 8-1, with G_d = 8,000 lb/in.

8.4.11.2 Strength Acceptance Criteria

Gypsum plaster has a yield capacity of approximately 400 pounds per linear foot.

8.4.11.3 Deformation Acceptance Criteria

See Section 8.4.5.3.

8.4.11.4 Connections

The tensile and bearing capacity of the plaster, rather than the connections, will often govern failure. The relatively low strength of this material makes connections between parts of the shear wall assembly and the other elements of the lateral-force-resisting system of secondary concern.

8.4.12 Gypsum Plaster on Gypsum Lath Shear Walls

8.4.12.1 Stiffness for Analysis

Gypsum plaster on gypsum lath is similar to gypsum wallboard (see Section 8.4.13 for a discussion of gypsum wallboard). The deflection of these shear walls can be determined using Equation 8-1, with G_d = 10,000 lb/in.

8.4.12.2 Strength Acceptance Criteria

These are similar to those for gypsum wallboard, with an approximate yield capacity of 80 pounds per linear foot. See Section 8.4.13.

8.4.12.3 Deformation Acceptance Criteria

See Section 8.4.5.3.

8.4.12.4 Connections

See Section 8.4.11.4.

8.4.13 Gypsum Wallboard Shear Walls

8.4.13.1 Stiffness for Analysis

Gypsum wallboard has a very low lateral-force-resistance capacity, but is relatively stiff until cracking occurs. These shear walls are suitable only where earthquake shear loads are very low. The deflection of these shear walls can be determined using Equation 8-1, with G_d = 8,000 lb/in.

8.4.13.2 Strength Acceptance Criteria

Gypsum wallboard has a yield capacity of approximately 100 pounds per linear foot. This capacity is for typical 7-inch nail spacing of 1/2-inch or 5/8-inch-thick panels with 4d or 5d nails. Higher capacities can be used if closer nail spacing, multilayers of gypsum board, and/or the presence of blocking at all panel edges is verified.

8.4.13.3 Deformation Acceptance Criteria

See Section 8.4.5.3.

8.4.13.4 Connections

See Section 8.4.11.4.

8.4.14 Gypsum Sheathing Shear Walls

8.4.14.1 Stiffness for Analysis

Gypsum sheathing is similar to gypsum wallboard (see Section 8.4.13 for a detailed discussion). The deflection of these shear walls can be determined using Equation 8-1, with G_d = 8,000 lb/in.

8.4.14.2 Strength Acceptance Criteria

These are similar to those for gypsum wallboard (see Section 8.4.13).

8.4.14.3 Deformation Acceptance Criteria

See Section 8.4.5.3.

8.4.14.4 Connections

See Section 8.4.11.4.

8.4.15 Plaster on Metal Lath Shear Walls

8.4.15.1 Stiffness for Analysis

Plaster on metal lath is similar to stucco but with less strength. Metal lath and plaster walls are stiff until cracking occurs. These shear walls are suitable only where earthquake shear loads are low. The deflection of these shear walls can be determined using Equation 8-1, with G_d = 12,000 lb/in.

8.4.15.2 Strength Acceptance Criteria

Plaster on metal lath has a yield capacity of approximately 150 pounds per linear foot.

8.4.15.3 Deformation Acceptance Criteria

See Section 8.4.5.3.

8.4.15.4 Connections

See Section 8.3.2.2B.

8.4.16 Horizontal Lumber Sheathing with Cut-In Braces or Diagonal Blocking Shear Walls

8.4.16.1 Stiffness for Analysis

This assembly is similar to horizontal sheathing without braces, except that the cut-in braces or diagonal blocking provide higher stiffness at initial loads. After the braces or blocking fail (at low loads), the behavior of the wall is the same as with horizontal sheathing without braces. See Section 8.4.4 for more information about horizontal sheathing.

8.4.16.2 Strength Acceptance Criteria

See Section 8.4.4.

8.4.16.3 Deformation Acceptance Criteria

See Section 8.4.5.3.

8.4.16.4 Connections

See Section 8.3.2.2B.

8.4.17 Fiberboard or Particleboard Sheathing Shear Walls

8.4.17.1 Stiffness for Analysis

Fiberboard sheathing is very weak, lacks stiffness, and is not able to resist lateral loads. Particleboard comes in two varieties: one is similar to structural panels, the other (nonstructural) is slightly stronger than gypsum board but more brittle. Fiberboard sheathing is not suitable for resisting lateral loads, and nonstructural particleboard should only be used to resist very low earthquake loads. For structural particleboard sheathing, see Section 8.4.9. The deflection of shear walls sheathed in nonstructural particleboard can be determined using Equation 8-1, with G_d = 6,000 lb/in.

8.4.17.2 Strength Acceptance Criteria

Fiberboard has very low strength. For structural particleboard, see the structural panel section (Section 8.4.9). Nonstructural particleboard has a yield capacity of approximately 100 pounds per linear foot.

8.4.17.3 Deformation Acceptance Criteria

See Section 8.4.5.3.

8.4.17.4 Connections

See Section 8.4.11.4.

8.4.18 Light Gage Metal Frame Shear Walls

8.4.18.1 Plaster on Metal Lath

See Section 8.4.15.

8.4.18.2 Gypsum Wallboard

See Section 8.4.13.

8.4.18.3 Plywood or Structural Panels

See Section 8.4.9. Refer to fastener manufacturer's data for allowable loads on fasteners. Yield capacity can be estimated by multiplying normal allowable load values for 2.8, or for allowable load values that are listed for wind or seismic loads, multiply by 2.1 to obtain estimated yield values.

8.5 Wood Diaphragms

The behavior of horizontal wood diaphragms is influenced by the type of sheathing, size, and amount of fasteners, presence of perimeter chord or flange members, and the ratio of span to depth of the diaphragm. Openings or penetrations through the diaphragm also effect the behavior and capacity of the diaphragm (see Section 8.5.11).

The expected capacity of the diaphragm, Q_{CE}, is determined from the yield shear capacity of the existing or enhanced diaphragm as described in Sections 8.5.2 through 8.5.9. For braced or horizontal truss type systems, the expected capacity, Q_{CE}, is determined from the member or connection yield capacity and conventional static truss analysis, as described in Section 8.5.10.

8.5.1 Types of Wood Diaphragms

8.5.1.1 Existing Wood Diaphragms

A. Single Straight Sheathed Diaphragms

Typically, these consist of 1" x sheathing laid perpendicular to the framing members; 2" x or 3" x sheathing may also be present. The sheathing serves the dual purpose of supporting gravity loads and resisting shear forces in the diaphragm. Most often, 1" x sheathing is nailed with 8d or 10d nails, with two or more nails at each sheathing board. Shear forces perpendicular to the direction of the sheathing are resisted by the nail couple. Shear forces parallel to the direction of the sheathing are transferred through the nails in the supporting joists or framing members below the sheathing joints.

B. Double Straight Sheathed Diaphragms

Construction is the same as that for single straight sheathed diaphragms, except that an upper layer of straight sheathing is laid over the lower layer of sheathing. The upper sheathing can be placed either perpendicular or parallel to the lower layer of sheathing. If the upper layer of sheathing is parallel to the lower layer, the board joints are usually offset sufficiently that nails at joints in the upper layer of sheathing are driven into a common sheathing board below, with sufficient edge distance. The upper layer of sheathing is nailed to the framing members through the lower layer of sheathing.

C. Single Diagonally Sheathed Wood Diaphragms

Typically, 1" x sheathing is laid at an approximate 45-degree angle to the framing members. In some cases 2" x sheathing may also be used. The sheathing supports gravity loads and resists shear forces in the diaphragm. Commonly, 1" x sheathing is nailed with 8d nails, with two or more nails per board. The recommended nailing for diagonally sheathed diaphragms is published in the *Western Woods Use Book* (WWPA, 1983) and UBC (ICBO, 1994a). The shear capacity of the diaphragm is dependent on the size and quantity of the nails at each sheathing board.

D. Diagonal Sheathing with Straight Sheathing or Flooring Above

Typically, these consist of a lower layer of 1" x diagonal sheathing laid at a 45-degree angle to the framing members, with a second layer of straight sheathing or wood flooring laid on top of the diagonal sheathing at a 90-degree angle to the framing members. Both layers of sheathing support gravity loads, and resist shear forces in the diaphragm. Sheathing boards are commonly nailed with 8d nails, with two or more nails per board.

E. Double Diagonally Sheathed Wood Diaphragms

Typically, these consist of a lower layer of 1" x diagonal sheathing with a second layer of 1" x diagonal sheathing laid at a 90-degree angle to the lower layer. The sheathing supports gravity loads and resists shear forces in the diaphragm. The sheathing is commonly nailed with 8d nails, with two or more nails per board. The recommended nailing for double diagonally sheathed diaphragms is published in the WWPA (1983).

F. Wood Structural Panel Sheathed Diaphragms

Typically, these consist of wood structural panels, such as plywood or oriented strand board, placed on framing members and nailed in place. Different grades and thicknesses of wood structural panels are commonly used, depending on requirements for gravity load support and shear capacity. Edges at the ends of the wood structural panels are usually supported by the framing members. Edges at the sides of the panels can be blocked or unblocked. In some cases, tongue and groove wood structural panels are used. Nailing patterns and nail size can vary greatly. Nail spacing is commonly in the range of 3 to 6 inches on center at the supported and blocked edges of the panels, and 10 to 12 inches on center at the panel infield. Staples are sometimes used to attach the wood structural panels.

G. Braced Horizontal Diaphragms

Typically, these consist of “X” rod bracing and wood struts forming a horizontal truss system at the floor or roof levels of the building. The “X” bracing usually consists of steel rods drawn taut by turnbuckles or nuts. The struts usually consist of wood members, which may or may not be part of the gravity-load-bearing system of the floor or roof. The steel rods function as tension members in the horizontal truss, while the struts function as compression members. Truss chords (similar to diaphragm chords) are needed to resist bending in the horizontal truss system.

8.5.1.2 Wood Diaphragms Enhanced for Rehabilitation

A. Wood Structural Panel Overlays on Straight or Diagonally Sheathed Diaphragms

Diaphragm shear capacity and stiffness can be increased by overlaying new wood structural panels over existing sheathed diaphragms. These diaphragms typically consist of new wood structural panels placed over existing straight or diagonal sheathing and nailed or stapled to the existing framing members through the existing sheathing. If the new overlay is nailed only to the existing framing members—without nailing at the panel edges perpendicular to the framing—the response of the new overlay will be similar to that of an unblocked wood structural panel diaphragm. Nails and staples should be of sufficient length to provide the required embedment into framing members below the sheathing.

If a stronger and stiffer diaphragm is desired, the joints of the new wood structural panel overlay can be placed parallel to the joints of the existing sheathing, with the overlay nailed or stapled to the existing sheathing. The edges of the new wood structural panels should be offset from the joints in the existing sheathing below by a sufficient distance that the new nails may be driven into the existing sheathing without splitting the sheathing. If the new panels are nailed at all edges as described above, the response of the new overlay will be similar to that of a blocked wood structural panel diaphragm. As an alternative, new blocking may be installed below all panel joints perpendicular to the existing framing members.

Because the joints of the overlay and the joints of the existing sheathing may not be offset consistently without cutting the panels, it may be advantageous to place the wood structural panel overlay at a 45-degree angle to the existing sheathing. If the existing diaphragm is straight sheathed, the new overlay should be placed at a 45-degree angle to the existing sheathing and joists. If the existing diaphragm is diagonally sheathed, the new wood structural panel overlay should be placed perpendicular to the existing joists at a 45-

degree angle to the diagonal sheathing. Nails should be driven into the existing sheathing with sufficient edge distance to prevent splitting of the existing sheathing. At boundaries, nails should be of sufficient length to penetrate through the sheathing into the framing below. New structural panel overlays shall be connected to the shear wall or vertical bracing elements to ensure the effectiveness of the added panel.

Care should be exercised when placing new wood structural panel overlays on existing diaphragms. The changes in stiffness and dynamic characteristics of the diaphragm may have negative effects by causing increased forces in other components or elements. The increased stiffness and the associated increase in dynamic forces may not be desirable in some diaphragms for certain Performance Levels.

B. Wood Structural Panel Overlays on Existing Wood Structural Panel Diaphragms

New wood structural panel overlays may be placed over existing wood structural panel diaphragms to strengthen and stiffen existing diaphragms. The placement of a new overlay over an existing diaphragm should follow the same construction methods and procedures as for straight and diagonally sheathed diaphragms (see Section 8.5.1.2A). Panel joints should be offset, or else the overlay should be placed at a 45-degree angle to the existing wood structural panels.

C. Increased Attachment

In some cases, existing diaphragms may be enhanced by increasing the nailing or attachment of the existing sheathing to the supporting framing. For straight sheathed diaphragms, the increase in shear capacity will be minimal. Double straight sheathed diaphragms with minimal nailing in the upper or both layers of sheathing may be enhanced significantly by adding new nails or staples to the existing diaphragm. The same is true for diaphragms that are single diagonally sheathed, double diagonally sheathed, or single diagonally sheathed with straight sheathing or flooring.

Plywood diaphragms can also be enhanced by increased nailing or attachment to the supporting framing and by adding blocking to the diaphragm at the plywood joints. In some cases, increased nailing at the plywood panel infield may also be required. If the required shear capacity and/or stiffness is greater than that which can be provided by increased attachment, a new overlay on the existing diaphragm may be required to provide the desired enhancement.

8.5.1.3 New Wood Diaphragms

A. Wood Structural Panel Sheathed Diaphragms

Typically, these consist of wood structural panels—such as plywood or oriented strand board—placed, nailed, or stapled in place on existing framing members after existing sheathing has been removed. Different grades and thicknesses of wood structural panels can be used, depending on the requirements for gravity load support and diaphragm shear capacity. In most cases, the panels are placed with the long dimension perpendicular to the framing members, and panel edges at the ends of the panels are supported by, and nailed to, the framing members. Edges at the sides of the panels can be blocked or unblocked, depending on the shear capacity and stiffness required in the new diaphragm. Wood structural panels can be placed in various patterns as shown in APA publications (APA, 1983) and various codes (e.g., ICBO, 1994a).

B. Single Diagonally Sheathed Wood Diaphragms

See Section 8.5.1.1C.

C. Double Diagonally Sheathed Wood Diaphragms

See Section 8.5.1.1E.

D. Braced Horizontal Diaphragms

See Section 8.5.1.1G. Because the special horizontal framing in the truss is an added structural feature, it is usually more economical to design floor or roof sheathing as a diaphragm in new construction, which eliminates the need for the “X” bracing and stronger wood members at the compression struts. Braced horizontal diaphragms are more feasible where sheathing cannot provide sufficient shear capacity, or where diaphragm openings reduce the shear capacity of the diaphragm and additional shear capacity is needed.

8.5.2 Single Straight Sheathed Diaphragms

8.5.2.1 Stiffness for Analysis

Straight sheathed diaphragms are characterized by high flexibility with a long period of vibration. These diaphragms are suitable for low shear conditions where control of diaphragm deflections is not needed to attain the desired Performance Levels. The deflection of straight sheathed diaphragms can be approximated using Equation 8-5:

$$\Delta = v\,L^4 / (G_d\, b^3) \qquad \text{(8-5)}$$

where:

b	=	Diaphragm width, ft
G_d	=	Diaphragm shear stiffness, lb/in.
L	=	Diaphragm span, ft between shear walls or collectors
v	=	Maximum shear in the direction under consideration, lb/ft
Δ	=	Calculated diaphragm deflection, in.

For straight sheathed diaphragms with or without chords, G_d = approximately 200,000 lb/in.

8.5.2.2 Strength Acceptance Criteria

Straight sheathed diaphragms have a low yield capacity of approximately 120 pounds per foot for chorded and unchorded diaphragms. The yield capacity for straight sheathed diaphragms is dependent on the size, number, and spacing between the nails at each sheathing board, and the spacing of the supporting framing members. The shear capacity of straight sheathed diaphragms can be calculated using the nail-couple method. See ATC (1981) for a discussion of calculating the shear capacity of straight sheathed diaphragms.

8.5.2.3 Deformation Acceptance Criteria

Deformation acceptance criteria will largely depend on the allowable deformations for other structural and nonstructural components and elements that are laterally supported by the diaphragm. Allowable deformations must also be consistent with the permissible damage state of the diaphragm. See Table 8-1 for m factors for use in Equation 3-18 for the LSP.

The coordinates for the normalized force-deflection curve for use in nonlinear procedures are shown in Table 8-2. The values in this table refer to Figure 8-1 in the following way. Distance d (see Figure 8-1) is considered the maximum deflection the diaphragm can undergo and still maintain its yield strength. Distance e is the maximum deflection at a reduced strength c.

8.5.2.4 Connections

The load capacity of connections between diaphragms and shear walls or other vertical elements, as well as diaphragm chords and shear collectors, is very important. These connections should have sufficient load capacity and ductility to deliver the required force to the vertical elements without sudden brittle failure in a connection or series of connections.

8.5.3 Double Straight Sheathed Wood Diaphragms

8.5.3.1 Stiffness for Analysis

The double sheathed system will provide a significant increase in stiffness over a single straight sheathed diaphragm, but very little test data is available on the stiffness and strength of these diaphragms. It is important that both layers of straight sheathing have sufficient nailing, and that the joints of the top layer are either offset or perpendicular to the bottom layer. The approximate deflection of double straight sheathed diaphragms can be calculated using Equation 8-5, with G_d as follows:

Double straight sheathing, chorded:	G_d = 1,500,000 lb/in.
Double straight sheathing, unchorded:	G_d = 700,000 lb/in.

8.5.3.2 Strength Acceptance Criteria

Typical yield shear capacity of double straight sheathed diaphragms is approximately 600 pounds per foot for chorded diaphragms. For unchorded diaphragms, the typical yield capacity is approximately 400 pounds per foot. The strength and stiffness of double straight sheathed diaphragms is highly dependent on the nailing of the upper layer of sheathing. If the upper layer has minimal nailing, the increase in strength and stiffness over a single straight sheathed diaphragm may be slight. If the upper layer of sheathing has nailing similar to that of the lower layer of sheathing, the increase in strength and stiffness will be significant.

8.5.3.3 Deformation Acceptance Criteria

See Section 8.5.2.3.

8.5.3.4 Connections

See Section 8.5.2.4.

8.5.4 Single Diagonally Sheathed Wood Diaphragms

8.5.4.1 Stiffness for Analysis

Single diagonally sheathed diaphragms are significantly stiffer than straight sheathed diaphragms, but are still quite flexible. The deflection of single diagonally sheathed diaphragms can be calculated using Equation 8-5, with G_d as follows:

Single diagonal sheathing, chorded:	G_d = 500,000 lb/in.
Single diagonal sheathing, unchorded:	G_d = 400,000 lb/in.

8.5.4.2 Strength Acceptance Criteria

Diagonally sheathed diaphragms are usually capable of resisting moderate shear loads. Typical yield shear capacity for diagonally sheathed wood diaphragms with chords is approximately 600 pounds per foot. Typical yield capacity for unchorded diaphragms is approximately 70% of the value for chorded diaphragms, or 420 pounds per foot. The shear capacity of diagonally sheathed diaphragms can be calculated based on the shear capacity of the nails in each of the sheathing boards. Because the diagonal sheathing boards function in tension and compression to resist shear forces in the diaphragm, and the boards are placed at a 45-degree angle to the chords at the ends of the diaphragm, the component of the force in the sheathing boards that is perpendicular to the axis of the end chords will create a bending force in the end chords. If the shear in diagonally sheathed diaphragms is limited to approximately 300 pounds per foot or less, bending forces in the end chords is usually neglected. If shear forces exceed 300 pounds per foot, the end chords should be designed or reinforced to resist bending forces from the sheathing. See ATC (1981) for methods of calculating the shear capacity of diagonally sheathed diaphragms.

8.5.4.3 Deformation Acceptance Criteria

See Section 8.5.2.3.

8.5.4.4 Connections

See Section 8.5.2.4.

8.5.5 Diagonal Sheathing with Straight Sheathing or Flooring Above Wood Diaphragms

8.5.5.1 Stiffness for Analysis

Straight sheathing or flooring over diagonal sheathing will provide a significant increase in stiffness over single sheathed diaphragms. The approximate deflection of diagonally sheathed diaphragms with straight sheathing or flooring above can be calculated using Equation 8-5, with G_d as follows:

Diagonal sheathing with straight sheathing, chorded:	G_d = 1,800,000 lb/in.
Diagonal sheathing with straight sheathing, unchorded:	G_d = 900,000 lb/in.

The increased stiffness of these diaphragms may make them suitable where Life Safety or Immediate Occupancy Performance Levels are desired.

8.5.5.2 Strength Acceptance Criteria

Shear capacity is dependent on the nailing of the diaphragm. Typical yield capacity for these diaphragms is approximately 900 pounds per foot for chorded diaphragms and approximately 625 pounds per foot for unchorded diaphragms. The strength and stiffness of diagonally sheathed diaphragms with straight sheathing above is highly dependent on the nailing of both layers of sheathing. Both layers of sheathing should have at least two 8d common nails at each support.

8.5.5.3 Deformation Acceptance Criteria

See Section 8.5.2.3.

8.5.5.4 Connections

See Section 8.5.2.4.

8.5.6 Double Diagonally Sheathed Wood Diaphragms

8.5.6.1 Stiffness for Analysis

Double diagonally sheathed diaphragms have greater stiffness than diaphragms with single diagonal sheathing. The response of these diaphragms is similar to the response of diagonally sheathed diaphragms with straight sheathing overlays. The approximate deflection of double diagonally sheathed diaphragms can be calculated using Equation 8-5, with G_d as follows:

Double diagonal sheathing, chorded:	G_d = 1,800,000 lb/in.
Double diagonal sheathing, unchorded:	G_d = 900,000 lb/in.

The increased stiffness of these diaphragms may make them suitable where Life Safety or Immediate Occupancy Performance Levels are desired.

8.5.6.2 Strength Acceptance Criteria

Shear capacity is dependent on the nailing of the diaphragm, but these diaphragms are usually suitable for moderate to high shear loads, and have a typical yield capacity of approximately 900 pounds per foot for chorded diaphragms and 625 pounds per foot for unchorded diaphragms. Yield shear capacities are similar to those of diagonally sheathed diaphragms with straight sheathing overlays. The sheathing boards in both layers of sheathing should be nailed with at least two 8d common nails. The presence of a double layer of diagonal sheathing will eliminate the bending forces that single diagonally sheathed diaphragms impose on the chords at the ends of the diaphragm. As a result, the bending capacity of the end chords does not have an effect on the shear capacity and stiffness of the diaphragm.

8.5.6.3 Deformation Acceptance Criteria

See Section 8.5.2.3.

8.5.6.4 Connections

See Section 8.5.2.4.

8.5.7 Wood Structural Panel Sheathed Diaphragms

8.5.7.1 Stiffness for Analysis

The response of wood structural panel sheathed diaphragms is dependent on the thickness of the wood structural panels, nailing pattern, and presence of chords in the diaphragm, as well as other factors. The deflection of blocked and chorded wood structural panel diaphragms with constant nailing across the diaphragm length can be determined using Equation 8-6:

$$\Delta_y = 5\,v_y\,L^3 / (8EAb) + v_y\,L / (4Gt) + 0.188\,L\,e_n + \Sigma(\Delta_c X) / (2b) \qquad (8\text{-}6)$$

where:

A = Area of chord cross section, in.^{2}

b = Diaphragm width, ft

E = Modulus of elasticity of diaphragm chords, psi

e_n = Nail deformation at yield load per nail, based on maximum shear per foot v_y divided by the number of nails per foot
For 8d nails, e_n = .06
For 10d nails, e_n = .04

G = Modulus of rigidity of wood structural panel, psi

L = Diaphragm span between shear walls or collectors, ft

t = Effective thickness of plywood for shear, in.

v_y = Yield shear in the direction under consideration, lb/ft

Δ_y = Calculated diaphragm deflection at yield, in.

$\Sigma(\Delta_c X)$ = Sum of individual chord-splice slip values on both sides of the diaphragm, each multiplied by its distance to the nearest support

The deflection of blocked and chorded wood structural panel diaphragms with variable nailing across the diaphragm length can be determined using Equation 8-7:

$$\Delta_y = 5\,v_y\,L^3 / (8EAb) + v_y\,L / (4Gt) + 0.376\,L\,e_n + \Sigma(\Delta_c X) / (2b) \qquad (8\text{-}7)$$

The deflection for unblocked diaphragms may be calculated using Equation 8-5, with diaphragm shear stiffness G_d as follows:

Unblocked, chorded diaphragms:	G_d = 800,000 lb/in.
Unblocked, unchorded diaphragms:	G_d = 400,000 lb/in.

8.5.7.2 Strength Acceptance Criteria

Shear capacities of wood structural panel diaphragms are primarily dependent on the nailing at the plywood panel edges, and the thickness and grade of the plywood in the diaphragm. The yield shear capacity, V_y, of wood structural panel diaphragms with chords can be calculated as follows:

If test data are available:	V_y = 0.8 x ultimate diaphragm shear value V_u
If test data are not available:	V_y = 2.1 x allowable diaphragm shear value (see ICBO, [1994a] and APA [1983])

For unchorded diaphragms, multiply the yield shear capacity for chorded diaphragms calculated above by 70%. Unchorded diaphragms with L/b > 3.5 are not considered to be effective for resisting lateral forces.

8.5.7.3 Deformation Acceptance Criteria

See Section 8.5.2.3.

8.5.7.4 Connections

See Section 8.5.2.4.

8.5.8 Wood Structural Panel Overlays on Straight or Diagonally Sheathed Diaphragms

8.5.8.1 Stiffness for Analysis

The stiffness of existing straight sheathed diaphragms can be increased significantly by placing a new plywood overlay over the existing diaphragm. The stiffness of existing diagonally sheathed diaphragms and plywood diaphragms will be increased, but not in proportion to the stiffness increase for straight sheathed diaphragms. Placement of the new wood structural panel overlay should be consistent with Section 8.5.1.2A. Depending on the nailing of the new overlay, the response of the diaphragm may be similar to that of a blocked or an unblocked diaphragm. The approximate deflection of wood structural panel overlays on straight or diagonally sheathed diaphragms can be calculated using Equation 8-5, with G_d as follows:

Unblocked, chorded diaphragm:	G_d = 900,000 lb/in.
Unblocked, unchorded diaphragm:	G_d = 500,000 lb/in.
Blocked, chorded diaphragm:	G_d = 1,800,000 lb/in.
Blocked, unchorded diaphragm:	G_d = 700,000 lb/in.

The increased stiffness of these diaphragms may make them suitable where Life Safety or Immediate Occupancy Performance Levels are desired.

8.5.8.2 Strength Acceptance Criteria

Typical yield capacity for diaphragms with a plywood overlay over existing straight and diagonal sheathing is approximately 450 pounds per foot for unblocked chorded diaphragms and approximately 300 pounds per foot for unblocked unchorded diaphragms. The yield capacity of blocked and chorded wood structural panel overlays over existing sheathing is approximately 65% of the ultimate shear capacity, or 2 times the allowable shear capacity of a comparable wood structural panel diaphragm without the existing sheathing below. The yield capacity of blocked and unchorded wood structural panel overlays over existing sheathing is approximately 50% of the ultimate shear capacity, or 1.5 times the allowable shear capacity of a comparable wood structural panel diaphragm without the existing sheathing below.

8.5.8.3 Deformation Acceptance Criteria

See Section 8.5.2.3.

8.5.8.4 Connections

See Section 8.5.2.4.

8.5.9 Wood Structural Panel Overlays on Existing Wood Structural Panel Diaphragms

8.5.9.1 Stiffness for Analysis

Diaphragm deflection shall be calculated according to Equation 8-6. Since two layers of plywood are present, the effective thickness of plywood t will be based on two layers of plywood. The nail slip e_n portion of the equation shall be adjusted for the increased nailing with two layers of plywood. Nail slip in the outer layer of plywood shall be increased 25% to account for the increased slip. It is important that nails in the upper layer of plywood have sufficient embedment in the framing to resist the required force and limit slip to the required level.

8.5.9.2 Strength Acceptance Criteria

Yield shear capacity for chorded diaphragms shall be calculated based on the combined two layers of plywood, using the methodology in Section 8.5.7.2. The yield shear capacity of the overlay should be limited to 75% of the values calculated using these procedures.

8.5.9.3 Deformation Acceptance Criteria

See Section 8.5.2.3.

8.5.9.4 Connections

See Section 8.5.2.4.

8.5.10 Braced Horizontal Diaphragms

8.5.10.1 Stiffness for Analysis

The stiffness and deflection of braced horizontal diaphragms can be determined using typical analysis techniques for trusses.

8.5.10.2 Strength Acceptance Criteria

The strength of the horizontal truss system can be determined using typical analysis techniques for trusses, and is dependent on the strength of the individual components and connections in the truss system. In many cases the capacity of the connections between truss components will be the limiting factor in the strength of the horizontal truss system.

8.5.10.3 Deformation Acceptance Criteria

Deformation acceptance criteria will largely depend on the allowable deformations for other structural and nonstructural components and elements that are laterally supported by the diaphragm. Allowable deformations must also be consistent with the desired damage state of the diaphragm. The individual *m* factors for the components and connections in the horizontal truss system listed in Table 8-1 will need to be applied in the truss analysis for the desired Performance Level.

8.5.10.4 Connections

The load capacity of connections between the members of the horizontal truss and shear walls or other vertical elements is very important. These connections must have sufficient load capacity and ductility to deliver the required force to the vertical elements without sudden brittle failure in a connection or series of connections. See Section 8.3.2.2B.

8.5.11 Effects of Chords and Openings in Wood Diaphragms

The presence of any but small openings in wood diaphragms will cause a reduction in the stiffness and yield capacity of the diaphragm, due to a reduced length of diaphragm available to resist lateral forces. Special analysis techniques and detailing are required at the openings. The presence or addition of chord members around the openings will reduce the loss in stiffness of the diaphragm and limit damage in the area of the openings. See ATC (1981) and APA (1983) for a discussion of the effects of openings in wood diaphragms.

The presence of chords at the perimeter of a diaphragm will significantly reduce the diaphragm deflection due to bending, and increase the stiffness of the diaphragm over that of an unchorded diaphragm. However, the increase in stiffness due to chords in a single straight sheathed diaphragm is minimal, due to the flexible nature of these diaphragms.

8.6 Wood Foundations

8.6.1 Wood Piling

Wood piles are generally used with a concrete pile cap, and simply key into the base of the concrete cap. The piles are usually treated with preservatives; they should be checked to determine whether deterioration has occurred and to verify the type of treatment. Piles are either friction- or end-bearing piles resisting only vertical loads. Piles are generally not able to resist uplift loads because of the manner in which they are attached to the pile cap. The piles may be subjected to lateral loads from seismic loading, which are resisted by bending of the piles. The analysis of pile bending is generally based on a pinned connection at the top of the pile, and fixity of the pile at some depth established by the geotechnical engineer. Maximum stress in the piles should be based on 2.8 time the values given in the *National Design Specification for Wood Construction, Part VI* (AF&PA, 1991a). Deflection of piles under seismic load can be calculated based on the assumed point of fixity. However, it should be evaluated with consideration for the approximate nature of the original assumption of the depth to point of fixity. Where battered piles are present, the lateral loads can be resisted by the horizontal component of the axial load.

For Immediate Occupancy, an *m* factor of 1.25 should be used in the analysis of the pile bending or axial force; for Life Safety, a factor of 2.5 can be used; and a factor of 3.0 can be used for Collapse Prevention.

8.6.2 Wood Footings

Wood grillage footings, sleepers, skids, and pressure-treated all-wood foundations are sometime encountered in existing structures. These footings should be thoroughly inspected for indications of deterioration, and replaced with reinforced concrete footings where possible. The seismic resistance for these types of footings is generally very low; they are essentially dependent on friction between the wood and soil for their performance.

8.6.3 Pole Structures

Pole structures resist lateral loads by acting as cantilevers fixed in the ground, with the lateral load considered to be applied perpendicular to the pole axis. It is possible to design pole structures to have moment-resisting capacity at floor and roof levels by the use of knee braces or trusses. Pole structures are frequently found on sloping sites. The varying unbraced lengths of the poles generally affect the stiffness and performance of the structure, and can result in unbalanced loads to the various poles along with significant torsional distortion, which must be investigated and evaluated. Added horizontal and diagonal braces can be used to reduce the flexibility of tall poles or reduce the torsional eccentricity of the structure.

8.6.3.1 Materials and Component Properties

The strength of the components, elements, and connections of a pole structure are the same as for a conventional structure. See Section 8.3 for recommendations.

8.6.3.2 Deformation Acceptance Criteria

Deformation characteristics are the same as for a member or frame that is subject to combined flexural and axial loads; pole structures are analyzed using conventional procedures.

8.6.3.3 Factors for the Linear Static Procedure

It is recommended that an *m* factor of 1.2 be used for a cantilevered pole structure for the Immediate Occupancy Performance Level, a value of 3.0 for Life Safety, and 3.5 for Collapse Prevention. Where concentrically braced diagonals are used or added to enhance the capacity of the structure, an *m* factor of 1.0 should be used for Immediate Occupancy, a value of 2.5 for Life Safety, and 3.0 for Collapse Prevention.

8.7 Definitions

Assembly: A collection of structural members and/or components connected in such a manner that load applied to any one component will affect the stress conditions of adjacent parallel components.

Aspect ratio: Ratio of height to width for vertical diaphragms, and width to depth for horizontal diaphragms.

Balloon framing: Continuous stud framing from sill to roof, with intervening floor joists nailed to studs and supported by a let-in ribbon. (See platform framing.)

Boundary component (boundary member): A member at the perimeter (edge or opening) of a shear wall or horizontal diaphragm that provides tensile and/or compressive strength.

Composite panel: A structural panel comprising thin wood strands or wafers bonded together with exterior adhesive.

Chord: See diaphragm chord.

Collector: See drag strut.

Condition of service: The environment to which the structure will be subjected. Moisture conditions are the most significant issue; however, temperature can have a significant effect on some assemblies.

Connection: A link between components or elements that transmits actions from one component or element to another component or element. Categorized by type of action (moment, shear, or axial), connection links are frequently nonductile.

Cripple wall: Short wall between foundation and first floor framing.

Cripple studs: Short studs between header and top plate at opening in wall framing or studs between base sill and sill of opening.

Decay: Decomposition of wood caused by action of wood-destroying fungi. The term “dry rot” is used interchangeably with decay.

Decking: Solid sawn lumber or glued laminated decking, nominally two to four inches thick and four inches and wider. Decking may be tongue-and-groove or connected at longitudinal joints with nails or metal clips.

Design resistance: Resistance (force or moment as appropriate) provided by member or connection; the product of adjusted resistance, the resistance factor, confidence factor, and time effect factor.

Diaphragm: A horizontal (or nearly horizontal) structural element used to distribute inertial lateral forces to vertical elements of the lateral-force-resisting system.

Diaphragm chord: A diaphragm component provided to resist tension or compression at the edges of the diaphragm.

Diaphragm ratio: See aspect ratio.

Diaphragm strut: See drag strut.

Dimensioned lumber: Lumber from nominal two through four inches thick and nominal two or more inches wide.

Dowel bearing strength: The maximum compression strength of wood or wood-based products when subjected to bearing by a steel dowel or bolt of specific diameter.

Dowel type fasteners: Includes bolts, lag screws, wood screws, nails, and spikes.

Drag strut: A component parallel to the applied load that collects and transfers diaphragm shear forces to the vertical lateral-force-resisting components or elements, or distributes forces within a diaphragm. Also called collector, diaphragm strut, or tie.

Dressed size: The dimensions of lumber after surfacing with a planing machine. Usually 1/2 to 3/4 inch less than nominal size.

Dry service: Structures wherein the maximum equilibrium moisture content does not exceed 19%.

Edge distance: The distance from the edge of the member to the center of the nearest fastener. When a member is loaded perpendicular to the grain, the loaded edge shall be defined as the edge in the direction toward which the fastener is acting.

Gauge or row spacing: The center-to-center distance between fastener rows or gauge lines.

Glulam beam: Shortened term for glued-laminated beam.

Grade: The classification of lumber in regard to strength and utility, in accordance with the grading rules of an approved agency.

Grading rules: Systematic and standardized criteria for rating the quality of wood products.

Gypsum wallboard or drywall: An interior wall surface sheathing material sometimes considered for resisting lateral forces.

Hold-down: Hardware used to anchor the vertical chord forces to the foundation or framing of the structure in order to resist overturning of the wall.

King stud: Full height stud or studs adjacent to openings that provide out-of-plane stability to cripple studs at openings.

Light framing: Repetitive framing with small uniformly spaced members.

Load duration: The period of continuous application of a given load, or the cumulative period of intermittent applications of load. (See time effect factor.)

Load/slip constant: The ratio of the applied load to a connection and the resulting lateral deformation of the connection in the direction of the applied load.

Load sharing: The load redistribution mechanism among parallel components constrained to deflect together.

LRFD (Load and Resistance Factor Design): A method of proportioning structural components (members, connectors, connecting elements, and assemblages) using load and resistance factors such that no applicable limit state is exceeded when the structure is subjected to all design load and resistance factor combinations.

Lumber: The product of the sawmill and planing mill, usually not further manufactured other than by sawing, resawing, passing lengthwise through a standard planing machine, crosscutting to length, and matching.

Lumber size: Lumber is typically referred to by size classifications. Additionally, lumber is specified by manufacturing classification. Rough lumber and dressed lumber are two of the routinely used manufacturing classifications.

Mat-formed panel: A structural panel designation representing panels manufactured in a mat-formed process, such as oriented strand board and waferboard.

Moisture content: The weight of the water in wood expressed as a percentage of the weight of the oven-dried wood.

Nominal size: The approximate rough-sawn commercial size by which lumber products are known and sold in the market. Actual rough-sawn sizes vary from the nominal. Reference to standards or grade rules is required to determine nominal to actual finished size relationships, which have changed over time.

Oriented strandboard: A structural panel comprising thin elongated wood strands with surface layers arranged in the long panel direction and core layers arranged in the cross panel direction.

Panel: A sheet-type wood product.

Panel rigidity or stiffness: The in-plane shear rigidity of a panel, the product of panel thickness and modulus of rigidity.

Panel shear: Shear stress acting through the panel thickness.

Particleboard: A panel manufactured from small pieces of wood, hemp, and flax, bonded with synthetic or organic binders, and pressed into flat sheets.

Pile: A deep structural component transferring the weight of a building to the foundation soils and resisting vertical and lateral loads; constructed of concrete, steel, or wood; usually driven into soft or loose soils.

Pitch or spacing: The longitudinal center-to-center distance between any two consecutive holes or fasteners in a row.

Planar shear: The shear that occurs in a plane parallel to the surface of a panel, which has the ability to cause the panel to fail along the plies in a plywood panel or in a random layer in a nonveneer or composite panel.

Platform framing: Construction method in which stud walls are constructed one floor at a time, with a floor or roof joist bearing on top of the wall framing at each level.

Ply: A single sheet of veneer, or several strips laid with adjoining edges that form one veneer lamina in a glued plywood panel.

Plywood: A structural panel comprising plies of wood veneer arranged in cross-aligned layers. The plies are bonded with an adhesive that cures upon application of heat and pressure.

Pole: A round timber of any size or length, usually used with the larger end in the ground.

Pole structure: A structure framed with generally round continuous poles that provide the primary vertical frame and lateral-load-resisting system.

Preservative: A chemical that, when suitably applied to wood, makes the wood resistant to attack by fungi, insects, marine borers, or weather conditions.

Pressure-preservative treated wood: Wood products pressure-treated by an approved process and preservative.

Primary (strong) panel axis: The direction that coincides with the length of the panel.

Punched metal plate: A light steel plate fastening having punched teeth of various shapes and configurations that are pressed into wood members to effect transfer shear. Used with structural lumber assemblies.

Required member resistance: Load effect (force, moment, stress, action as appropriate) acting on an element or connection, determined by structural analysis from the factored loads and the critical load combinations.

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

Resistance factor: A reduction factor applied to member resistance that accounts for unavoidable deviations of the actual strength from the nominal value, and the manner and consequences of failure.

Row of fasteners: Two or more fasteners aligned with the direction of load.

Rough lumber: Lumber as it comes from the saw prior to any dressing operation.

Subdiaphragm: A portion of a larger diaphragm used to distribute loads between members.

Seasoned lumber: Lumber that has been dried. Seasoning takes place by open-air drying within the limits of moisture contents attainable by this method, or by controlled air drying (i.e., kiln drying).

Sheathing: Lumber or panel products that are attached to parallel framing members, typically forming wall, floor, ceiling, or roof surfaces.

Shrinkage: Reduction in the dimensions of wood due to a decrease of moisture content.

Structural-use panel: A wood-based panel product bonded with an exterior adhesive, generally 4' x 8' or larger in size. Included under this designation are plywood, oriented strand board, waferboard, and composite panels. These panel products meet the requirements of PS 1-95 (NIST, 1995) or PS 2-92 (NIST, 1992) and are intended for structural use in residential, commercial, and industrial applications.

Stud: Wood member used as vertical framing member in interior or exterior walls of a building, usually 2" x 4" or 2" x 6" sizes, and precision end-trimmed.

Tie: See drag strut.

Tie-down: Hardware used to anchor the vertical chord forces to the foundation or framing of the structure in order to resist overturning of the wall.

Timbers: Lumber of nominal five or more inches in smaller cross-section dimension.

Time effect factor: A factor applied to adjusted resistance to account for effects of duration of load. (See load duration.)

Waferboard: A nonveneered structural panel manufactured from two- to three-inch flakes or wafers bonded together with a phenolic resin and pressed into sheet panels.

8.8 Symbols

This list may exclude symbols appearing once only, when defined at that appearance.

E	Young's modulus of elasticity of chord members
G	Modulus of rigidity of wood structural panel
G_d	Modulus of rigidity of diaphragm
h	Height of wall
h/L	Aspect ratio
L	Length of wall or floor/roof diaphragm
L/b	Diaphragm ratio
V	Shear to element or component
V_y	Shear to element at yield
b	Depth of floor/roof horizontal diaphragm
b	Diaphragm width, ft
e_n	Nail deformation at yield load level
v	Shear per foot
v_y	Shear per foot at yield
Δ	Deflection of diaphragm or bracing element, in.
Δ_y	Deflection of diaphragm or bracing element at yield

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9. Seismic Isolation and Energy Dissipation (Systematic Rehabilitation)

Chapter 9 includes detailed guidelines for building rehabilitation with seismic (and base) isolation and passive energy dissipation systems, and limited guidance for other systems such as active energy dissipation devices.

The basic form and formulation of guidelines for seismic isolation and energy dissipation systems have been established and coordinated with the Rehabilitation Objectives, Performance Levels, and seismic ground shaking hazard criteria of Chapter 2 and the linear and nonlinear procedures of Chapter 3.

Criteria for modeling the stiffness, strength, and deformation capacities of conventional structural components of buildings with seismic isolation or energy dissipation systems are given in Chapters 5 through 8 and Chapter 10.

Seismic Isolation and Energy Dissipation as Rehabilitation Strategies

Seismic isolation and energy dissipation systems are viable design strategies that have already been used for seismic rehabilitation of a number of buildings. Other special seismic protective systems—including active control, hybrid combinations of active and passive energy devices, and tuned mass and liquid dampers—may also provide practical solutions in the near future. These systems are similar in that they enhance performance during an earthquake by modifying the building's response characteristics.

Seismic isolation and energy dissipation systems will not be appropriate design strategies for most buildings, particularly buildings that have only Limited Rehabilitation Objectives. In general, these systems will be most applicable to the rehabilitation of buildings whose owners desire superior earthquake performance and can afford the special costs associated with the design, fabrication, and installation of seismic isolators and/or energy dissipation devices. These costs are typically offset by the reduced need for stiffening and strengthening measures that would otherwise be required to meet Rehabilitation Objectives.

Seismic isolation and energy dissipation systems are relatively new and sophisticated concepts that require more extensive design and detailed analysis than do most conventional rehabilitation schemes. Similarly, design (peer) review is required for all rehabilitation schemes that use either seismic isolation or energy dissipation systems.

9.1 Introduction

This chapter provides guidelines for the application of special seismic protective systems to building rehabilitation. Specific guidance is provided for seismic (base) isolation systems in Section 9.2 and for passive energy dissipation systems in Section 9.3. Section 9.4 provides additional, limited guidance for other special seismic systems, including active control systems, hybrid active and passive systems, and tuned mass and liquid dampers.

Special seismic protective systems should be evaluated as possible rehabilitation strategies based on the Rehabilitation Objectives established for the building. Prior to implementation of the guidelines of this chapter, the user should establish the following criteria as presented in Chapter 2:

- The Rehabilitation Objective for the building
  - Performance Level
  - Seismic Ground Shaking Hazard

Seismic isolation and energy dissipation systems include a wide variety of concepts and devices. In most cases, these systems and devices will be implemented with some additional conventional strengthening of the structure; in all cases they will require evaluation of existing building elements. As such, this chapter supplements the guidelines of other chapters of this document with additional criteria and methods of analysis that are appropriate for buildings rehabilitated with seismic isolators and/or energy dissipation devices.

Seismic isolation is increasingly becoming considered for historic buildings that are free-standing and have a basement or bottom space of no particular historic significance. In selecting such a solution, special consideration should be given to the possibility that historic or archaeological resources may be present at the site. If that is determined, the guidance of the State Historic Preservation Officer should be obtained in a timely manner. Isolation is also often considered for essential facilities, to protect valuable contents, and on buildings with a complete, but insufficiently strong lateral-force-resisting system.

9.2 Seismic Isolation Systems

This section specifies analysis methods and design criteria for seismic isolation systems that are based on the Rehabilitation Objectives, Performance Levels, and Seismic Ground Shaking Hazard criteria of Chapter 2.

The methods described in this section augment the analysis requirements of Chapter 3. The analysis methods and other criteria of this section are based largely on the 1994 *NEHRP Provisions* (BSSC, 1995) for new buildings, augmented with changes proposed by Technical Subcommittee 12 of the Provisions Update Committee of the Building Seismic Safety Council for the 1997 *NEHRP Provisions* (BSSC, 1997).

9.2.1 Background

Buildings rehabilitated with a seismic isolation system may be thought of as composed of three distinct segments: the structure above the isolation system, the isolation system itself, and the foundation and other structural elements below the isolation system.

The isolation system includes wind-restraint and tie-down systems, if such systems are required by these *Guidelines*. The isolation system also includes supplemental energy dissipation devices, if such devices are used to transmit force between the structure above the isolation system and the structure below the isolation system.

This section provides guidance primarily for the design, analysis, and testing of the isolation system and for determination of seismic load on structural elements and nonstructural components. Criteria for rehabilitation of structural elements other than the isolation system, and criteria for rehabilitation of nonstructural components, should follow the applicable guidelines of other chapters of this document, using loads and deformations determined by the procedures of this section.

Seismic Isolation Performance Objectives

Seismic isolation has typically been used as a Rehabilitation Strategy that enhances the performance of the building above that afforded by conventional stiffening and strengthening schemes. Seismic isolation rehabilitation projects have targeted performance at least equal to, and commonly exceeding, the Basic Safety Objective of these *Guidelines*, effectively achieving Immediate Occupancy or better performance.

A number of buildings rehabilitated with seismic isolators have been historic. For these projects, seismic isolation reduced the extent and intrusion of seismic modifications on the historical fabric of the building that would otherwise be required to meet desired Performance Levels.

See the *Commentary* for detailed discussions on the development of isolation provisions for new buildings (Section C9.2.1.1) and the design philosophy on which the provisions are based (Section C9.2.1.2). The *Commentary* also provides an overview of seismic isolation rehabilitation projects (Section C9.2.1.3) and goals (Section C9.2.1.4).

9.2.2 Mechanical Properties and Modeling of Seismic Isolation Systems

9.2.2.1 General

A seismic isolation system is the collection of all individual seismic isolators (and separate wind restraint and tie-down devices, if such devices are used to meet the requirements of these *Guidelines*). Seismic isolation systems may be composed entirely of one type of seismic isolator, a combination of different types of seismic isolators, or a combination of seismic isolators acting in parallel with energy dissipation devices (i.e., a hybrid system).

Seismic isolators are classified as either elastomeric, sliding, or other isolators. Elastomeric isolators are typically made of layers of rubber separated by steel shims. Elastomeric isolators may be any one of the following: high-damping rubber bearings (HDR), low-damping rubber bearings (RB) or low-damping rubber bearings with a lead core (LRB). Sliding isolators may be flat assemblies or have a curved surface, such as the friction-pendulum system (FPS). Rolling systems may be characterized as a subset of sliding systems. Rolling isolators may be flat assemblies or have a curved or conical surface, such as the ball and cone system (BNC). Other isolators are not discussed.

This section provides guidance for modeling of elastomeric isolators and sliding isolators. Guidance for modeling of energy dissipation devices may be found in Section 9.3. Information on hybrid systems is provided in the *Commentary* (Section C9.2.2.2C)

9.2.2.2 Mechanical Properties of Seismic Isolators

A. Elastomeric Isolators

The mechanical characteristics of elastomeric isolators should be known in sufficient detail to establish force-deformation response properties and their dependence, if any, on axial-shear interaction, bilateral deformation, load history (including the effects of "scragging" of virgin elastomeric isolators; that is, the process of subjecting an elastomeric bearing to one or more cycles of large amplitude displacement), temperature, and other environmental loads and aging effects (over the design life of the isolator).

For the purpose of mathematical modeling of isolators, mechanical characteristics may be based on analysis and available material test properties, but verification of isolator properties used for design should be based on tests of isolator prototypes, as described in Section 9.2.9.

B. Sliding Isolators

The mechanical characteristics of sliding isolators should be known in sufficient detail to establish force-deformation response properties and their dependence, if any, on contact pressure, rate of loading (velocity), bilateral deformation, temperature, contamination, and other environmental loads and aging effects (over the design life of the isolator).

For the purpose of mathematical modeling of isolators, mechanical characteristics may be based on analysis and available material test properties, but verification of isolator properties used for design should be based on tests of isolator prototypes, as described in Section 9.2.9.

9.2.2.3 Modeling of Isolators

A. General

If the mechanical characteristics of a seismic isolator are dependent on design parameters such as axial load (due to gravity, earthquake overturning effects, and vertical earthquake shaking), rate of loading (velocity), bilateral deformation, temperature, or aging, then upper- and lower-bound values of stiffness and damping should be used to determine the range and sensitivity of response to design parameters.

B. Linear Models

Linear procedures use effective stiffness, k_{eff}, and effective damping, β_{eff}, to characterize nonlinear properties of isolators. The restoring force of an isolator is calculated as the product of effective stiffness, k_{eff}, and response displacement, D:

$$F = k_{eff}D \tag{9-1}$$

The effective stiffness, k_{eff}, of an isolator is calculated from test data using Equation 9-12. Similarly, the area enclosed by the force-displacement hysteresis loop is used to calculate the effective damping, β_{eff}, of an isolator using Equation 9-13. Both effective stiffness and effective damping are, in general, amplitude-dependent and should be evaluated at all response displacements of design interest.

C. Nonlinear Models

Nonlinear procedures should explicitly model the nonlinear force-deflection properties of isolators.

Damping should be modeled explicitly by inelastic (hysteretic) response of isolators. Additional viscous damping should not be included in the model unless supported by rate-dependent tests of isolators.

9.2.2.4 Isolation System and Superstructure Modeling

A. General

Mathematical models of the isolated building—including the isolation system, the lateral-force-resisting system and other structural components and elements, and connections between the isolation system and the structure above and below the isolation system—should conform to the requirements of Chapters 2 and 3 and the guidelines given below.

B. Isolation System Model

The isolation system should be modeled using deformational characteristics developed and verified by test in accordance with the requirements of Section 9.2.9.

The isolation system should be modeled with sufficient detail to:

1. Account for the spatial distribution of isolator units
2. Calculate translation, in both horizontal directions, and torsion of the structure above the isolation interface, considering the most disadvantageous location of mass eccentricity
3. Assess overturning/uplift forces on individual isolators
4. Account for the effects of vertical load, bilateral load, and/or the rate of loading, if the force deflection properties of the isolation system are dependent on one or more of these factors
5. Assess forces due to P-Δ moments

C. Superstructure Model

The maximum displacement of each floor, and the total design displacement and total maximum displacement across the isolation system, should be calculated using a model of the isolated building that incorporates the force-deflection characteristics of nonlinear components, and elements of the isolation system and the superstructure.

Isolation systems with nonlinear components include, but are not limited to, systems that do not meet the criteria of Section 9.2.3.3A Item (2).

Lateral-force-resisting systems with nonlinear components and elements include, but are not limited to, systems described by both of the following criteria.

1. For all deformation-controlled actions, Equation 3-18 is satisfied using a value of *m* equal to 1.0.
2. For all force-controlled actions, Equation 3-19 is satisfied.

Design forces and displacements in primary components of the lateral-force-resisting system may be calculated using a linearly elastic model of the isolated structure, provided the following criteria are met.

1. Pseudo-elastic properties assumed for nonlinear isolation system components are based on the maximum effective stiffness of the isolation system.
2. The lateral-force-resisting system remains essentially linearly elastic for the earthquake demand level of interest.

9.2.3 General Criteria for Seismic Isolation Design

9.2.3.1 General

Criteria for the seismic isolation of buildings are divided into two sections:

1. Rehabilitation of the building
2. Design, analysis, and testing of the isolation system

A. Basis For Design

Seismic Rehabilitation Objectives of the building should be consistent with those set forth in Chapter 2. The design, analysis, and testing of the isolation system should be based on the guidelines of this chapter.

B. Stability of the Isolation System

The stability of the vertical-load-carrying components of the isolation system should be verified by analysis and test, as required, for a lateral displacement equal to the total maximum displacement, or for the maximum displacement allowed by displacement-restraint devices, if such devices are part of the isolation system.

C. Configuration Requirements

The regularity of the isolated building should be designated as being either regular or irregular on the basis of the structural configuration of the structure above the isolation system.

9.2.3.2 Ground Shaking Criteria

Ground shaking criteria are required for the design earthquake, which is user-specified and may be chosen equal to the BSE-1, and for the Maximum Considered Earthquake (MCE), equal to the BSE-2, as described in Chapter 2.

A. User-Specified Design Earthquake

For the design earthquake, the following ground shaking criteria should be established:

1. Short period spectral response acceleration parameter, S_{DS}, and spectral response acceleration parameter at 1.0 second, S_{D1}
2. Five-percent-damped response spectrum of the design earthquake (when a response spectrum is required for linear procedures by Section 9.2.3.3A, or to define acceleration time histories)
3. At least three acceleration time histories compatible with the design earthquake spectrum (when acceleration time histories are required for nonlinear procedures by Section 9.2.3.3B)

B. Maximum Earthquake

For the BSE-2, the following ground shaking criteria should be established:

1. Short period spectral response acceleration parameter, S_{MS}, and spectral response acceleration parameter at 1.0 second, S_{M1}
2. Five-percent-damped site-specific response spectrum of the BSE-2 (when a response spectrum is required for linear procedures by Section 9.2.3.3A, or to define acceleration time histories)
3. At least three acceleration time histories compatible with the BSE-2 spectrum (when acceleration time histories are required for nonlinear procedures by Section 9.2.3.3B)

9.2.3.3 Selection of Analysis Procedure

A. Linear Procedures

Linear procedures may be used for design of seismically isolated buildings, provided the following criteria are met.

1. The building is located on Soil Profile Type A, B, C, or D; or E (if $S_1 \geq 0.6$ for BSE-2).
2. The isolation system meets all of the following criteria:
   a. The effective stiffness of the isolation system at the design displacement is greater than one-third of the effective stiffness at 20% of the design displacement.
   b. The isolation system is capable of producing a restoring force as specified in Section 9.2.7.2D.
   c. The isolation system has force-deflection properties that are essentially independent of the rate of loading.
   d. The isolation system has force-deflection properties that are independent of vertical load and bilateral load.
   e. The isolation system does not limit BSE-2 displacement to less than S_{M1}/S_{D1} times the total design displacement.
3. The structure above the isolation system remains essentially linearly elastic for the BSE-2.

Response spectrum analysis should be used for design of seismically-isolated buildings that meet any of the following criteria.

- The building is over 65 feet (19.8 meters) in height.
- The effective period of the structure, T_M, is greater than three seconds.
- The effective period of the isolated structure, T_D, is less than or equal to three times the elastic, fixed-base period of the structure above the isolation system.
- The structure above the isolation system is irregular in configuration.

B. Nonlinear Procedures

Nonlinear procedures should be used for design of seismic-isolated buildings for which the following conditions apply.

1. The structure above the isolation system is nonlinear for the BSE-2.
2. The building is located on Soil Profile Type E (if $S_1 > 0.6$ for BSE-2) or Soil Profile Type F.
3. The isolation system does not meet all of the criteria of Section 9.2.3.3A, Item (2).

Nonlinear acceleration time history analysis is required for the design of seismically isolated buildings for which conditions (1) and (2) apply.

9.2.4 Linear Procedures

9.2.4.1 General

Except as provided in Section 9.2.5, every seismically isolated building, or portion thereof, should be designed and constructed to resist the earthquake displacements and forces specified by this section.

9.2.4.2 Deformation Characteristics of the Isolation System

The deformation characteristics of the isolation system should be based on properly substantiated tests performed in accordance with Section 9.2.9.

The deformation characteristics of the isolation system should explicitly include the effects of the wind-restraint and tie-down systems, and supplemental energy-dissipation devices, if such a systems and devices are used to meet the design requirements of these guidelines.

9.2.4.3 Minimum Lateral Displacements

A. Design Displacement

The isolation system should be designed and constructed to withstand, as a minimum, lateral earthquake displacements that act in the direction of each of the main horizontal axes of the structure in accordance with the equation:

$$D_D = \left[\frac{g}{4\pi^2}\right]\frac{S_{D1}T_D}{B_{D1}} \qquad (9\text{-}2)$$

B. Effective Period at the Design Displacement

The effective period, T_D, of the isolated building at the design displacement should be determined using the deformational characteristics of the isolation system in accordance with the equation:

$$T_D = 2\pi\sqrt{\frac{W}{K_{Dmin}g}} \qquad (9\text{-}3)$$

C. Maximum Displacement

The maximum displacement of the isolation system, D_M, in the most critical direction of horizontal response should be calculated in accordance with the equation:

$$D_M = \left[\frac{g}{4\pi^2}\right]\frac{S_{M1}T_M}{B_{M1}} \qquad (9\text{-}4)$$

D. Effective Period at the Maximum Displacement

The effective period, T_M, of the isolated building at the maximum displacement should be determined using the deformational characteristics of the isolation system in accordance with the equation:

$$T_M = 2\pi\sqrt{\frac{W}{K_{Mmin}g}} \qquad (9\text{-}5)$$

E. Total Displacement

The total design displacement, D_{TD}, and the total maximum displacement, D_{TM}, of components of the isolation system should include additional displacement due to actual and accidental torsion calculated considering the spatial distribution of the effective stiffness of the isolation system at the design displacement and the most disadvantageous location of mass eccentricity.

The total design displacement, D_{TD}, and the total maximum displacement, D_{TM}, of components of an isolation system with a uniform spatial distribution of effective stiffness at the design displacement should be taken as not less than that prescribed by the equations:

$$D_{TD} = D_D\left[1 + y\frac{12e}{b^2 + d^2}\right] \qquad (9\text{-}6)$$

$$D_{TM} = D_M\left[1 + y\frac{12e}{b^2 + d^2}\right] \qquad (9\text{-}7)$$

The total maximum displacement, D_{TM}, may be taken as less than the value prescribed by Equation 9-7, but not less than 1.1 times D_M, provided the isolation system is shown by calculation to be configured to resist torsion accordingly.

9.2.4.4 Minimum Lateral Forces

A. Isolation System and Structural Components and Elements at or below the Isolation System

The isolation system, the foundation, and all other structural components and elements below the isolation system should be designed and constructed to withstand a minimum lateral seismic force, V_b, prescribed by the equation:

$$V_b = K_{Dmax} D_D \tag{9-8}$$

B. Structural Components and Elements above the Isolation System

The components and elements above the isolation system should be designed and constructed to resist a minimum lateral seismic force, V_s, taken as equal to the value of V_b, prescribed by Equation 9-8.

C. Limits on V_s

The value of V_s should be taken as not less than the following:

1. The base shear corresponding to the design wind load
2. The lateral seismic force required to fully activate the isolation system factored by 1.5 (e.g., the yield level of a softening system, the ultimate capacity of a sacrificial wind-restraint system, or the break-away friction level of a sliding system factored by 1.5)

D. Vertical Distribution of Force

The total force should be distributed over the height of the structure above the isolation interface as follows:

$$F_x = \frac{V_s w_x h_x}{\sum_{i}^{n} w_i h_i} \tag{9-9}$$

At each level designated as x, the force F_x should be applied over the area of the building in accordance with the weight, w_x, distribution at that level, h_x. Response of structural components and elements should be calculated as the effect of the force F_x applied at the appropriate levels above the base.

9.2.4.5 Response Spectrum Analysis

A. Earthquake Input

The design earthquake spectrum should be used to calculate the total design displacement of the isolation system and the lateral forces and displacements of the isolated building. The BSE-2 spectrum should be used to calculate the total maximum displacement of the isolation system.

B. Modal Damping

Response Spectrum Analysis should be performed, using a damping value for isolated modes equal to the effective damping of the isolation system, or 30% of critical, whichever is less. The damping value assigned to higher modes of response should be consistent with the material type and stress level of the superstructure.

C. Combination of Earthquake Directions

Response Spectrum Analysis used to determine the total design displacement and total maximum displacement should include simultaneous excitation of the model by 100% of the most critical direction of ground motion, and not less than 30% of the ground motion in the orthogonal axis. The maximum displacement of the isolation system should be calculated as the vector sum of the two orthogonal displacements.

D. Scaling of Results

If the total design displacement determined by Response Spectrum Analysis is found to be less than the value of D_{TD} prescribed by Equation 9-6, or if the total maximum displacement determined by response spectrum analysis is found to be less than the value of D_{TM} prescribed by Equation 9-7, then all response parameters, including components actions and deformations, should be adjusted upward proportionally to the D_{TD} value, or the D_{TM} value, and used for design.

9.2.4.6 Design Forces and Deformations

Components and elements of the building should be designed for forces and displacements estimated by linear procedures using the acceptance criteria of Section 3.4.2.2, except that deformation-controlled components and elements should be designed using a component demand modifier no greater than 1.5.

9.2.5 Nonlinear Procedures

Isolated buildings evaluated using nonlinear procedures should be represented by three-dimensional models that incorporate both the nonlinear characteristics of the isolation system and the structure above the isolation system.

9.2.5.1 Nonlinear Static Procedure

A. General

The Nonlinear Static Procedure (NSP) for seismically isolated buildings should be based on the nonlinear procedure guidelines of Section 3.3.3, except that the target displacement and pattern of applied lateral load should be based on the criteria given in the following sections.

B. Target Displacement

In each principal direction, the building model should be pushed to the design earthquake target displacement, D'_D, and to the BSE-2 target displacement, D'_M, as defined by the following equations:

$$D'_D = \frac{D_D}{\sqrt{1 + \left(\frac{T_e}{T_D}\right)^2}} \quad (9\text{-}10)$$

$$D'_M = \frac{D_M}{\sqrt{1 + \left(\frac{T_e}{T_M}\right)^2}} \quad (9\text{-}11)$$

where T_e is the effective period of the superstructure on a fixed base as prescribed by Equation 3-10. The target displacements, D'_D and D'_M, should be evaluated at a control node that is located at the center of mass of the first floor above the isolation interface.

C. Lateral Load Pattern

The pattern of applied lateral load should be proportional to the distribution of the product of building mass and the deflected shape of the isolated mode of response at target displacement.

9.2.5.2 Nonlinear Dynamic Procedure

A. General

The Nonlinear Dynamic Procedure (NDP) for seismically isolated buildings should be based on the nonlinear procedure guidelines of Section 3.3.4, except that results should be scaled for design based on the criteria given in the following section.

B. Scaling of Results

If the design displacement determined by Time-History Analysis is found to be less than the value of D'_D prescribed by Equation 9-10, or if the maximum displacement determined by Response Spectrum Analysis is found to be less than the value of D'_M prescribed by Equation 9-11, then all response parameters, including component actions and deformations, should be adjusted upward proportionally to the D'_D value or the D'_M value, and used for design.

9.2.5.3 Design Forces and Deformations

Components and elements of the building should be designed for the forces and deformations estimated by nonlinear procedures using the acceptance criteria of Section 3.4.3.2.

9.2.6 Nonstructural Components

9.2.6.1 General

Parts or portions of a seismically isolated building, permanent nonstructural components and the attachments to them, and the attachments for permanent equipment supported by a building should be designed to resist seismic forces and displacements as given in this section and the applicable requirements of Chapter 11.

9.2.6.2 Forces and Displacements

A. Components and Elements at or above the Isolation Interface

Components and elements of seismically isolated buildings and nonstructural components, or portions thereof, that are at or above the isolation interface, should be designed to resist a total lateral seismic force equal to the maximum dynamic response of the element or component under consideration.

EXCEPTION: Elements of seismically isolated structures and nonstructural components, or portions thereof, may be designed to resist total lateral seismic force as required for conventional fixed-base buildings by Chapter 11.

B. Components and Elements That Cross the Isolation Interface

Elements of seismically isolated buildings and nonstructural components, or portions thereof, that cross the isolation interface should be designed to withstand the total maximum (horizontal) displacement and maximum vertical displacement of the isolation system at the total maximum (horizontal) displacement. Components and elements that cross the isolation interface should not restrict displacement of the isolated building or otherwise compromise the Rehabilitation Objectives of the building.

C. Components and Elements Below the Isolation Interface

Components and elements of seismically isolated buildings and nonstructural components, or portions thereof, that are below the isolation interface should be designed and constructed in accordance with the requirements of Chapter 11.

9.2.7 Detailed System Requirements

9.2.7.1 General

The isolation system and the structural system should comply with the general requirements of Chapter 2 and the requirements of Chapters 4 through 8. In addition, the isolation system and the structural system should comply with the detailed system requirements of this section.

9.2.7.2 Isolation System

A. Environmental Conditions

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

B. Wind Forces

Isolated buildings should resist design wind loads at all levels above the isolation interface in accordance with the applicable wind design provisions. At the isolation interface, a wind-restraint system should be provided to limit lateral displacement in the isolation system to a value equal to that required between floors of the structure above the isolation interface.

C. Fire Resistance

Fire resistance rating for the isolation system should be consistent with the requirements of columns, walls, or other such elements of the building.

D. Lateral Restoring Force

The isolation system should be configured to produce either a restoring force such that the lateral force at the total design displacement is at least $0.025W$ greater than the lateral force at 50% of the total design displacement, or a restoring force of not less than $0.05W$ at all displacements greater than 50% of the total design displacement.

EXCEPTION: The isolation system need not be configured to produce a restoring force, as required above, provided the isolation system is capable of remaining stable under full vertical load and accommodating a total maximum displacement equal to the greater of either 3.0 times the total design displacement or 36 S_{M1} inches.

E. Displacement Restraint

The isolation system may be configured to include a displacement restraint that limits lateral displacement due to the BSE-2 to less than S_{M1}/ S_{D1} times the total design displacement, provided that the seismically isolated building is designed in accordance with the following criteria when more stringent than the requirements of Section 9.2.3.

1. BSE-2 response is calculated in accordance with the dynamic analysis requirements of Section 9.2.5, explicitly considering the nonlinear characteristics of the isolation system and the structure above the isolation system.

2. The ultimate capacity of the isolation system, and structural components and elements below the isolation system, should exceed the force and displacement demands of the BSE-2.

3. The structure above the isolation system is checked for stability and ductility demand of the BSE-2.

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

F. Vertical Load Stability

Each component of the isolation system should be designed to be stable under the full maximum vertical load, $1.2Q_D + Q_L + |Q_E|$, and the minimum vertical load, $0.8Q_D - |Q_E|$, at a horizontal displacement equal to the total maximum displacement. The earthquake vertical load on an individual isolator unit, Q_E, should be based on peak building response due to the BSE-2.

G. Overturning

The factor of safety against global structural overturning at the isolation interface should be not less than 1.0 for required load combinations. All gravity and seismic loading conditions should be investigated. Seismic forces for overturning calculations should be based on the BSE-2, and the vertical restoring force should be based on the building's weight, *W*, above the isolation interface.

Local uplift of individual components and elements is permitted, provided the resulting deflections do not cause overstress or instability of the isolator units or other building components and elements. A tie-down system may be used to limit local uplift of individual components and elements, provided that the seismically isolated building is designed in accordance with the following criteria when more stringent than the requirements of Section 9.2.3.

1. BSE-2 response is calculated in accordance with the dynamic analysis requirements of Section 9.2.5, explicitly considering the nonlinear characteristics of the isolation system and the structure above the isolation system.

2. The ultimate capacity of the tie-down system should exceed the force and displacement demands of the BSE-2.

3. The isolation system is both designed to be stable and shown by test to be stable (Section 9.2.9.2F) for BSE-2 loads that include additional vertical load due to the tie-down system.

H. Inspection and Replacement

Access for inspection and replacement of all components and elements of the isolation system should be provided.

I. Manufacturing Quality Control

A manufacturing quality control testing program for isolator units should be established by the engineer responsible for the structural design.

9.2.7.3 Structural System

A. Horizontal Distribution of Force

A horizontal diaphragm or other structural components and elements should provide continuity above the isolation interface. The diaphragm or other structural components and elements should have adequate strength and ductility to transmit forces (due to nonuniform ground motion) from one part of the building to another, and have sufficient stiffness to effect rigid diaphragm response above the isolation interface.

B. Building Separations

Minimum separations between the isolated building and surrounding retaining walls or other fixed obstructions should be not less than the total maximum displacement.

9.2.8 Design and Construction Review

9.2.8.1 General

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

9.2.8.2 Isolation System

Isolation system design and construction review should include, but not be limited to, the following:

1. Site-specific seismic criteria, including site-specific spectra and ground motion time history, and all other design criteria developed specifically for the project

2. Preliminary design, including the determination of the total design and total maximum displacement of the isolation system, and the lateral force design level

3. Isolation system prototype testing (Section 9.2.9)

4. Final design of the isolated building and supporting analyses

5. Isolation system quality control testing (Section 9.2.7.2I)

9.2.9 Isolation System Testing and Design Properties

9.2.9.1 General

The deformation characteristics and damping values of the isolation system used in the design and analysis of seismically isolated structures should be based on the following tests of a selected sample of the components prior to construction.

The isolation system components to be tested should include isolators, and components of the wind restraint system and supplemental energy dissipation devices if such components and devices are used in the design.

The tests specified in this section establish design properties of the isolation system, and should not be considered as satisfying the manufacturing quality control testing requirements of Section 9.2.7.2I.

9.2.9.2 Prototype Tests

A. General

Prototype tests should be performed separately on two full-size specimens of each type and size of isolator of the isolation system. The test specimens should include components of the wind restraint system, as well as individual isolators, if such components are used in the design. Supplementary energy dissipation devices should be tested in accordance with Section 9.3.8 criteria. Specimens tested should not be used for construction unless approved by the engineer responsible for the structural design.

B. Record

For each cycle of tests, the force-deflection and hysteretic behavior of the test specimen should be recorded.

C. Sequence and Cycles

The following sequence of tests should be performed for the prescribed number of cycles at a vertical load equal to the average $Q_D + 0.5Q_L$ on all isolators of a common type and size:

1. Twenty fully reversed cycles of loading at a lateral force corresponding to the wind design force
2. Three fully reversed cycles of loading at each of the following displacements: $0.25D_D$, $0.50D_D$, $1.0D_D$, and $1.0D_M$
3. Three fully reversed cycles at the total maximum displacement, $1.0D_{TM}$
4. $30S_{D1}/S_{DS}B_D$, but not less than 10, fully reversed cycles of loading at the design displacement, $1.0D_D$

If an isolator is also a vertical-load-carrying element, then Item 2 of the sequence of cyclic tests specified above should be performed for two additional vertical load cases:

1. $1.2Q_D + 0.5Q_L + |Q_E|$
2. $0.8Q_D - |Q_E|$

where D, L, and E refer to dead, live, and earthquake loads. Q_D and Q_L are as defined in Section 3.2.8. The vertical test load on an individual isolator unit should include the load increment Q_E due to earthquake overturning, and should be equal to or greater than the peak earthquake vertical force response corresponding to the test displacement being evaluated. In these tests, the combined vertical load should be taken as the typical or average downward force on all isolators of a common type and size.

D. Isolators Dependent on Loading Rates

If the force-deflection properties of the isolators are dependent on the rate of loading, then each set of tests specified in Section 9.2.9.2C should be performed dynamically at a frequency equal to the inverse of the effective period, T_D, of the isolated structure.

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

The force-deflection properties of an isolator should be considered to be dependent on the rate of loading if there is greater than a plus or minus 10% difference in the effective stiffness at the design displacement (1) when tested at a frequency equal to the inverse of the effective period of the isolated structure and (2) when tested at any frequency in the range of 0.1 to 2.0 times the inverse of the effective period of the isolated structure.

E. Isolators Dependent on Bilateral Load

If the force-deflection properties of the isolators are dependent on bilateral load, then the tests specified in Sections 9.2.9.2C and 9.2.9.2D should be augmented to include bilateral load at the following increments of the total design displacement: 0.25 and 1.0; 0.50 and 1.0; 0.75 and 1.0; and 1.0 and 1.0.

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

The force-deflection properties of an isolator should be considered to be dependent on bilateral load, if the bilateral and unilateral force-deflection properties have greater than a plus or minus 15% difference in effective stiffness at the design displacement.

F. Maximum and Minimum Vertical Load

Isolators that carry vertical load should be statically tested for the maximum and minimum vertical load, at the total maximum displacement. In these tests, the combined vertical loads of $1.2Q_D + 1.0Q_L + |Q_E|$ should be taken as the maximum vertical force, and the combined vertical load of $0.8Q_D - |Q_E|$ should be taken as the minimum vertical force, on any one isolator of a common type and size. The earthquake vertical load on an individual isolator, Q_E, should be based on peak building response due to the BSE-2.

G. Sacrificial Wind-Restraint Systems

If a sacrificial wind-restraint system is part of the isolation system, then the ultimate capacity should be established by test.

H. Testing Similar Units

Prototype tests are not required if an isolator unit is:

1. Of similar dimensional characteristics
2. Of the same type and materials, and
3. Fabricated using identical manufacturing and quality control procedures

9.2.9.3 Determination of Force-Deflection Characteristics

The force-deflection characteristics of the isolation system should be based on the cyclic load testing of isolator prototypes specified in Section 9.2.9.2C.

As required, the effective stiffness of an isolator unit, k_{eff}, should be calculated for each cycle of deformation by the equation:

$$k_{eff} = \frac{|F^+| + |F^-|}{|\Delta^+| + |\Delta^-|} \qquad (9\text{-}12)$$

where F^+ and F^- are the positive and negative forces at positive and negative test displacements, Δ^+ and Δ^-, respectively.

As required, the effective damping of an isolator unit, β_{eff}, should be calculated for each cycle of deformation by the equation:

$$\beta_{eff} = \frac{2}{\pi}\left[\frac{E_{Loop}}{k_{eff}(|\Delta^+| + |\Delta^-|)^2}\right] \qquad (9\text{-}13)$$

where the energy dissipated per cycle of loading, E_{Loop}, and the effective stiffness, k_{eff}, are based on test displacements, Δ^+ and Δ^-.

9.2.9.4 System Adequacy

The performance of the test specimens should be assessed as adequate if the following conditions are satisfied.

1. The force-deflection plots of all tests specified in Section 9.2.9.2 have a nonnegative incremental force-carrying capacity.
2. For each increment of test displacement specified in Section 9.2.9.2C, Item (2), and for each vertical load case specified in Section 9.2.9.2C the following criteria are met.

   a. There is no greater than a plus or minus 15% difference between the effective stiffness at each of the three cycles of test and the average value of effective stiffness for each test specimen.

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

3. For each specimen there is no greater than a plus or minus 20% change in the initial effective stiffness of each test specimen over the $30S_{D1}/S_{DS}B_D$, but not less than 10, cycles of the test specified in Section 9.2.9.2C, Item (3).

4. For each specimen there is no greater than a 20% decrease in the initial effective damping over the $30S_{D1}/S_{DS}B_D$, but not less than 10, cycles of the test specified in Section 9.2.9.2C, Item (4).

5. All specimens of vertical-load-carrying elements of the isolation system remain stable at the total maximum displacement for static load as prescribed in Section 9.2.9.2F.

6. The effective stiffness and effective damping of test specimens fall within the limits specified by the engineer responsible for structural design.

9.2.9.5 Design Properties of the Isolation System

A. Maximum and Minimum Effective Stiffness

At the design displacement, the maximum and minimum effective stiffness of the isolation system, K_{Dmax} and K_{Dmin}, should be based on the cyclic tests of Section 9.2.9.2 and calculated by the formulas:

$$K_{Dmax} = \frac{\sum |F_D^+|_{max} + \sum |F_D^-|_{max}}{2D_D} \tag{9-14}$$

$$K_{Dmin} = \frac{\sum |F_D^+|_{min} + \sum |F_D^-|_{min}}{2D_D} \tag{9-15}$$

At the maximum displacement, the maximum and minimum effective stiffness of the isolation system should be based on cyclic tests of Section 9.2.9.2 and calculated by the formulas:

$$K_{Mmax} = \frac{\sum |F_M^+|_{max} + \sum |F_M^-|_{max}}{2D_M} \tag{9-16}$$

$$K_{Mmin} = \frac{\sum |F_M^+|_{min} + \sum |F_M^-|_{min}}{2D_M} \tag{9-17}$$

For isolators that are found by the tests of Sections 9.2.9.2C, 9.2.9.2D, and 9.2.9.2E to have force-deflection characteristics that vary with vertical load, rate of loading, or bilateral load, respectively, the values of K_{Dmax} and K_{Mmax} should be increased and the values of K_{Dmin} and K_{Mmin} should be decreased, as necessary, to bound the effects of the measured variation in effective stiffness.

B. Effective Damping

At the design displacement, the effective damping of the isolation system, β_D, should be based on the cyclic tests of Section 9.2.9.2 and calculated by the formula:

$$\beta_D = \frac{1}{2\pi}\left\lfloor \frac{\sum E_D}{K_{Dmax} D_D^2} \right\rfloor \tag{9-18}$$

In Equation 9-18, the total energy dissipated in the isolation system per displacement cycle, ΣE_D, should be taken as the sum of the energy dissipated per cycle in all isolators measured at test displacements, Δ^+ and Δ^-, that are equal in magnitude to the design displacement, D_D.

At the maximum displacement, the effective damping of the isolation system, β_M, should be based on the cyclic tests of Section 9.2.9.2 and calculated by the formula:

$$\beta_M = \frac{1}{2\pi}\left\lfloor \frac{\sum E_M}{K_{Mmax} D_M^2} \right\rfloor \tag{9-19}$$

In Equation 9-19, the total energy dissipated in the isolation system per displacement cycle, ΣE_M, should be taken as the sum of the energy dissipated per cycle in all isolators measured at test displacements, Δ^+ and Δ^-, that are equal in magnitude to the maximum displacement, D_M.

9.3 Passive Energy Dissipation Systems

This section specifies analysis methods and design criteria for energy dissipation systems that are based on the Rehabilitation Objectives, Performance Levels, and Seismic Ground Shaking Hazard criteria of Chapter 2.

9.3.1 General Requirements

This section provides guidelines for the implementation of passive energy dissipation devices in the seismic rehabilitation of buildings. In addition to the requirements provided herein, every rehabilitated building incorporating energy dissipation devices should be designed in accordance with the applicable provisions of the remainder of the *Guidelines* unless modified by the requirements of this section.

The energy dissipation devices should be designed with consideration given to other environmental conditions including wind, aging effects, creep, fatigue, ambient temperature, operating temperature, and exposure to moisture or damaging substances.

The building height limitations should not exceed the values for the structural system into which the energy dissipation devices are implemented.

The mathematical model of a rehabilitated building should include the plan and vertical distribution of the energy dissipation devices. Analysis of the mathematical model should account for the dependence of the devices on excitation frequency, ambient and operating temperature, velocity, sustained loads, and bilateral loads. Multiple analyses of the building may be necessary to capture the effects of varying mechanical characteristics of the devices.

Energy dissipation devices shall be capable of sustaining larger displacements (and velocities for velocity-dependent devices) than the maxima calculated in the BSE-2. The increase in displacement (and velocity) capacity is dependent on the level of redundancy in the supplemental damping system as follows:

1. If four or more energy dissipation devices are provided in a given story of a building, in one principal direction of the building, with a minimum of two devices located on each side of the center of stiffness of the story in the direction under consideration, all energy dissipation devices shall be capable of sustaining displacements equal to 130% of the maximum calculated displacement in the device in the BSE-2. A velocity-dependent device (see Section 9.3.3) shall also be capable of sustaining the force associated with a velocity equal to 130% of the maximum calculated velocity for that device in the BSE-2.

2. If fewer than four energy dissipation devices are provided in a given story of a building, in one principal direction of the building, or fewer than two devices are located on each side of the center of stiffness of the story in the direction under consideration, all energy dissipation devices shall be capable of sustaining displacements equal to 200% of the maximum calculated displacement in the device in the BSE-2. A velocity-dependent device shall also be capable of sustaining the force associated with a velocity equal to 200% of the maximum calculated velocity for that device in the BSE-2.

Energy Dissipation Performance Levels

Passive energy dissipation is an emerging technology that enhances the performance of the building by adding damping (and in some cases stiffness) to the building. The primary use of energy dissipation devices is to reduce earthquake displacement of the structure. Energy dissipation devices will also reduce force in the structure—provided the structure is responding elastically—but would not be expected to reduce force in structures that are responding beyond yield.

For most applications, energy dissipation provides an alternative approach to conventional stiffening and strengthening schemes, and would be expected to achieve comparable Performance Levels. In general, these devices would be expected to be good candidates for projects that have a Performance Level of Life Safety, or perhaps Immediate Occupancy, but would be expected to have only limited applicability to projects with a Performance Level of Collapse Prevention.

Other objectives may also influence the decision to use energy dissipation devices, since these devices can also be useful for control of building response due to small earthquakes, wind, or mechanical loads.

The components and connections transferring forces between the energy dissipation devices shall be designed to remain linearly elastic for the forces described in items 1 or 2 above—dependent upon the degree of redundancy in the supplemental damping system.

9.3.2 Implementation of Energy Dissipation Devices

The following subsections of Section 9.3 provide guidance to the design professional to aid in the implementation of energy dissipation devices. Guidelines and criteria for analysis procedures and component acceptance can be found in other chapters of the *Guidelines*.

Restrictions on the use of linear procedures are established in Chapter 2. These restrictions also apply to the linear procedures of Section 9.3.4. Restrictions on the use of nonlinear procedures, established in Chapter 2, also apply to the nonlinear procedures of Section 9.3.5. Example applications of linear and nonlinear procedures are provided in the *Commentary,* Section C9.3.9 (there is no corresponding section in the *Guidelines*).

9.3.3 Modeling of Energy Dissipation Devices

Energy dissipation devices are classified in this section as either displacement-dependent, velocity-dependent, or other. Displacement-dependent devices may exhibit either rigid-plastic (friction devices), bilinear (metallic yielding devices), or trilinear hysteresis. The response of displacement-dependent devices should be independent of velocity and/or frequency of excitation. Velocity-dependent devices include solid and fluid viscoelastic devices, and fluid viscous devices. The third classification (other) includes all devices that cannot be classified as either displacement- or velocity-dependent. Examples of “other” devices include shape-memory alloys (superelastic effect), friction-spring assemblies with recentering capability, and fluid restoring force-damping devices.

Models of the energy dissipation system should include the stiffness of structural components that are part of the load path between energy dissipation devices and the ground, if the flexibility of these components is significant enough to affect the performance of the energy dissipation system. Structural components whose flexibility could affect the performance of the energy dissipation system include components of the foundation, braces that work in series with the energy dissipation devices, and connections between braces and the energy dissipation devices.

Energy dissipation devices should be modeled as described in the following subsections, unless more advanced methods or phenomenological models are used.

9.3.3.1 Displacement-Dependent Devices

The force-displacement response of a displacement-dependent device is primarily a function of the relative displacement between each end of the device. The response of such a device is substantially independent of the relative velocity between each end of the device, and/or frequency of excitation.

Displacement-dependent devices should be modeled in sufficient detail so as to capture their force-displacement response adequately, and their dependence, if any, on axial-shear-flexure interaction, or bilateral deformation response.

For the purposes of evaluating the response of a displacement-dependent device from testing data, the force in a displacement-dependent device may be expressed as:

$$F = k_{eff}D \tag{9-20}$$

where the effective stiffness k_{eff} of the device is calculated as:

$$k_{eff} = \frac{|F^+| + |F^-|}{|D^+| + |D^-|} \tag{9-21}$$

and where forces in the device, F^+ and F^-, are evaluated at displacements D^+ and D^-, respectively.

9.3.3.2 Velocity-Dependent Devices

The force-displacement response of a velocity-dependent device is primarily a function of the relative velocity between each end of the device.

A. Solid Viscoelastic Devices

The cyclic response of viscoelastic solids is generally dependent on the frequency and amplitude of the motion, and the operating temperature (including temperature rise due to excitation).

Solid viscoelastic devices may be modeled using a spring and dashpot in parallel (Kelvin model). The spring and dashpot constants selected should adequately capture the frequency and temperature dependence of the device consistent with fundamental frequency of the rehabilitated building (f_1), and the operating temperature range. If the cyclic response of a viscoelastic solid device cannot be adequately captured by single estimates of the spring and dashpot constants, the response of the rehabilitated building should be estimated by multiple analyses of the building frame, using limited values for the spring and dashpot constants.

The force in a viscoelastic device may be expressed as:

$$F = k_{eff}D + C\dot{D} \tag{9-22}$$

where C is the damping coefficient for the viscoelastic device, D is the relative displacement between each end of the device, $\dot{D}$ is the relative velocity between each end of the device, and k_{eff} is the effective stiffness of the device calculated as:

$$k_{eff} = \frac{|F^+| + |F^-|}{|D^+| + |D^-|} = K' \tag{9-23}$$

where K' is the so-called storage stiffness.

The damping coefficient for the device should be calculated as:

$$C = \frac{W_D}{\pi\omega_1 D_{ave}^2} = \frac{K''}{\omega_1} \tag{9-24}$$

where K'' is the loss stiffness, the angular frequency ω_1 is equal to $2\pi f_1$, D_{ave} is the average of the absolute values of displacements D^+ and D^-, and W_D is the area enclosed by one complete cycle of the force-displacement response of the device.

B. Fluid Viscoelastic Devices

The cyclic response of viscoelastic fluid devices is generally dependent on the frequency and amplitude of the motion, and the operating temperature (including temperature rise due to excitation).

Fluid viscoelastic devices may be modeled using a spring and dashpot in series (Maxwell model). The spring and dashpot constants selected should adequately capture the frequency and temperature dependence of the device consistent with fundamental frequency of the rehabilitated building (f_1), and the operating temperature range. If the cyclic response of a viscoelastic fluid device cannot be adequately captured by single estimates of the spring and dashpot constants, the response of the rehabilitated building should be estimated by multiple analyses of the building frame, using limiting values for the spring and dashpot constants.

C. Fluid Viscous Devices

The cyclic response of a fluid viscous device is dependent on the velocity of motion; may be dependent on the frequency and amplitude of the motion; and is generally dependent on the operating temperature (including temperature rise due to excitation). Fluid viscous devices may exhibit some stiffness at high frequencies of cyclic loading. Linear fluid viscous dampers exhibiting stiffness in the frequency range 0.5 f_1 to 2.0 f_1 should be modeled as a fluid viscoelastic device.

In the absence of stiffness in the frequency range 0.5 f_1 to 2.0 f_1, the force in the fluid viscous device may be expressed as:

$$F = C_0|\dot{D}|^{\alpha}\ \mathrm{sgn}(\dot{D}) \tag{9-25}$$

where C_0 is the damping coefficient for the device, α is the velocity exponent for the device, $\dot{D}$ is the relative velocity between each end of the device, and sgn is the signum function that, in this case, defines the sign of the relative velocity term.

9.3.3.3 Other Types of Devices

Energy dissipation devices not classified as either displacement-dependent or velocity-dependent should be modeled using either established principles of mechanics or phenomenological models. Such models should accurately describe the force-velocity-displacement response of the device under all sources of loading (e.g., gravity, seismic, thermal).

9.3.4 Linear Procedures

Linear procedures are only permitted if it can be demonstrated that the framing system exclusive of the energy dissipation devices remains essentially linearly elastic for the level of earthquake demand of interest after the effects of added damping are considered. Further, the effective damping afforded by the energy dissipation shall not exceed 30% of critical in the fundamental mode. Other limits on the use of linear procedures are presented below.

The secant stiffness of each energy dissipation device, calculated at the maximum displacement in the device, shall be included in the mathematical model of the rehabilitated building. For the purpose of evaluating the regularity of a building, the energy dissipation devices shall be included in the mathematical model.

9.3.4.1 Linear Static Procedure

A. Displacement-Dependent Devices

The Linear Static Procedure (LSP) may be used to implement displacement-dependent energy dissipation devices, provided that the following requirements are satisfied:

1. The ratio of the maximum resistance in each story, in the direction under consideration, to the story shear demand calculated using Equations 3-7 and 3-8, shall range between 80% and 120% of the average value of the ratio for all stories. The maximum story resistance shall include the contributions from all components, elements, and energy dissipation devices.

2. The maximum resistance of all energy dissipation devices in a story, in the direction under consideration, shall not exceed 50% of the resistance of the remainder of the framing where said resistance is calculated at the displacements anticipated in the BSE-2. Aging and environmental effects shall be considered in calculating the maximum resistance of the energy dissipation devices.

   The pseudo lateral load of Equation 3-6 should be reduced by the damping modification factors of Table 2-15 to account for the energy dissipation (damping) afforded by the energy dissipation devices. The calculation of the damping effect should be estimated as:

$$\beta_{eff} = \beta + \frac{\sum_j W_j}{4\pi W_k} \tag{9-26}$$

where β is the damping in the framing system and is set equal to 0.05 unless modified in Section 2.6.1.5, W_j is work done by device j in one complete cycle corresponding to floor displacements δ_i, the summation extends over all devices j, and W_k is the maximum strain energy in the frame, determined using Equation 9-27:

$$W_k = \frac{1}{2}\sum_i F_i \delta_i \tag{9-27}$$

where F_i is the inertia force at floor level i and the summation extends over all floor levels.

B. Velocity-Dependent Devices

The LSP may be used to implement velocity-dependent energy dissipation devices provided that the following requirements are satisfied:

- The maximum resistance of all energy dissipation devices in a story, in the direction under consideration, shall not exceed 50% of the resistance of the remainder of the framing where said resistance is calculated at the displacements anticipated in the BSE-2. Aging and environmental effects shall be considered in calculating the maximum resistance of the energy dissipation devices.

The pseudo lateral load of Equation 3-6 should be reduced by the damping modification factors of Table 2-15 to account for the energy dissipation (damping) afforded by the energy dissipation devices. The calculation of the damping effect should be estimated as:

$$\beta_{eff} = \beta + \frac{\sum_j W_j}{4\pi W_k} \tag{9-28}$$

where β is the damping in the structural frame and is set equal to 0.05 unless modified in Section 2.6.1.5, W_j is work done by device j in one complete cycle corresponding to floor displacements δ_i, the summation extends over all devices j, and W_k is the maximum strain energy in the frame, determined using Equation 9-27.

The work done by linear viscous device j in one complete cycle of loading may be calculated as:

$$W_j = \frac{2\pi^2}{T} C_j \delta_{rj}^2 \tag{9-29}$$

where T is the fundamental period of the rehabilitated building including the stiffness of the velocity-dependent devices, C_j is the damping constant for device j, and δ_{rj} is the relative displacement between the ends of device j along the axis of device j. An alternative equation for calculating the effective damping of Equation 9-28 is:

$$\beta_{eff} = \beta + \frac{T \sum_j C_j \cos^2\theta_j \, \phi_{rj}^2}{\pi \sum_i \left(\frac{w_i}{g}\right) \phi_i^2} \tag{9-30}$$

where θ_j is the angle of inclination of device j to the horizontal, ϕ_{rj} is the first mode relative displacement between the ends of device j in the horizontal direction, w_i is the reactive weight of floor level i, ϕ_i is the first mode displacement at floor level i, and other terms are as defined above. Equation 9-30 applies to linear viscous devices only.

The design actions for components of the rehabilitated building should be calculated in three distinct stages of deformation as follows. The maximum action should be used for design.

1. **At the stage of maximum drift.** The lateral forces at each level of the building should be calculated using Equations 3-7 and 3-8, where V is the modified equivalent base shear.

2. **At the stage of maximum velocity and zero drift.** The viscous component of force in each energy dissipation device should be calculated by Equations 9-22 or 9-25, where the relative velocity $\dot{D}$ is given by $2\pi f_1 D$, where D is the relative displacement between the ends of the device calculated at the stage of maximum drift. The calculated viscous forces should be applied to the mathematical model of the building at the points of attachment of the devices and in directions consistent with the deformed shape of the building at maximum drift. The horizontal inertia forces at each floor level of the building should be applied concurrently with the viscous forces so that the horizontal displacement of each floor level is zero.

3. **At the stage of maximum floor acceleration.** Design actions in components of the rehabilitated building should be determined as the sum of [actions determined at the stage of maximum drift] times [CF_1] and [actions determined at the stage of maximum velocity] times [CF_2], where

$$CF_1 = \cos[\tan^{-1}(2\beta_{eff})] \tag{9-31}$$

$$CF_2 = \sin[\tan^{-1}(2\beta_{eff})] \tag{9-32}$$

in which β_{eff} is defined by either Equation 9-28 or Equation 9-30.

9.3.4.2 Linear Dynamic Procedure

The Linear Dynamic Procedures (LDP) of Section 3.3.2.2 should be followed unless explicitly modified by this section.

The response spectrum method of the LDP may by used when the effective damping in the fundamental mode of the rehabilitated building, in each principal direction, does not exceed 30% of critical.

A. Displacement-Dependent Devices

Application of the LDP for the analysis of rehabilitated buildings incorporating displacement-dependent devices is subject to the restrictions set forth in Section 9.3.4.1A.

For analysis by the Response Spectrum Method, the 5%-damped response spectrum may be modified to account for the damping afforded by the displacement-dependent energy dissipation devices. The 5%-damped acceleration spectrum should be reduced by the modal-dependent damping modification factor, B, either B_s or B_1, for periods in the vicinity of the mode under consideration; note that the value of B will be different for each mode of vibration. The damping modification factor in each significant mode should be determined using Table 2-15 and the calculated effective damping in that mode. The effective damping should be determined using a procedure similar to that described in Section 9.3.4.1A.

If the maximum base shear force calculated by dynamic analysis is less than 80% of the modified equivalent base shear of Section 9.3.4.1, component and element actions and deformations shall be proportionally increased to correspond to 80% of the modified equivalent base shear.

B. Velocity-Dependent Devices

For analysis by the Response Spectrum Method, the 5%-damped response spectrum may be modified to account for the damping afforded by the velocity-dependent energy dissipation devices. The 5%-damped acceleration spectrum should be reduced by the modal-dependent damping modification factor, B, either B_s or B_1, for periods in the vicinity of the mode under consideration; note that the value of B will be different for each mode of vibration. The damping modification factor in each significant mode should be determined using Table 2-15 and the calculated effective damping in that mode.

The effective damping in the m-th mode of vibration (β_{eff-m}) shall be calculated as:

$$\beta_{eff-m} = \beta_m + \frac{\sum_j W_{mj}}{4\pi W_{mk}} \tag{9-33}$$

where β_m is the m-th mode damping in the building frame, W_{mj} is work done by device j in one complete cycle corresponding to modal floor displacements δ_{mi}, and W_{mk} is the maximum strain energy in the frame in the m-th mode, determined using Equation 9-34:

$$W_{mk} = \frac{1}{2}\sum_i F_{mi}\delta_{mi} \tag{9-34}$$

where F_{mi} is the m-th mode horizontal inertia force at floor level i and δ_{mi} is the m-th mode horizontal displacement at floor level i. The work done by linear viscous device j in one complete cycle of loading in the m-th mode may be calculated as:

$$W_{mj} = \frac{2\pi^2}{T_m} C_j \delta_{mrj}^2 \tag{9-35}$$

where T_m is the m-th mode period of the rehabilitated building including the stiffness of the velocity-dependent devices, C_j is the damping constant for device j, and δ_{mrj} is the m-th mode relative displacement between the ends of device j along the axis of device j.

Direct application of the Response Spectrum Method will result in member actions at maximum drift. Member actions at maximum velocity and maximum acceleration in each significant mode should be determined using the procedure described in Section 9.3.4.1B. The combination factors CF_1 and CF_2 should be determined from Equations 9-31 and 9-32 using β_{eff-m} for the m-th mode.

If the maximum base shear force calculated by dynamic analysis is less than 80% of the modified equivalent base shear of Section 9.3.4.2, component and element actions and deformations shall be proportionally increased to correspond to 80% of the modified equivalent base shear.

9.3.5 Nonlinear Procedures

Subject to the limits set forth in Chapter 2, the nonlinear procedures of Section 3.3.3 may be used to implement passive energy dissipation devices without restriction.

9.3.5.1 Nonlinear Static Procedure

The Nonlinear Static Procedure (NSP) of Section 3.3.3 should be followed unless explicitly modified by this section.

The nonlinear mathematical model of the rehabilitated building should explicitly include the nonlinear force-velocity-displacement characteristics of the energy dissipation devices, and the mechanical characteristics of the components supporting the devices. Stiffness characteristics should be consistent with the deformations corresponding to the target displacement and a frequency equal to the inverse of period T_e as defined in Section 3.3.3.2.

The nonlinear mathematical model of the rehabilitated building shall include the nonlinear force-velocity-displacement characteristics of the energy dissipation

Benefits of Adding Energy Dissipation Devices

The benefit of adding *displacement-dependent* energy dissipation devices is recognized in the *Guidelines* by the increase in building stiffness afforded by such devices, and the reduction in target displacement associated with the reduction in T_e. The alternative Nonlinear Static Procedure, denoted in the *Commentary* as Method 2, uses a different strategy to calculate the target displacement and explicitly recognizes the added damping provided by the energy dissipation devices.

The benefits of adding *velocity-dependent* energy dissipation devices are recognized by the increases in stiffness and equivalent viscous damping in the building frame. For most velocity-dependent devices, the primary benefit will be due to the added viscous damping. Higher-mode damping forces in the energy dissipation devices must be evaluated regardless of the Nonlinear Static Procedure used—refer to the *Commentary* for additional information.

devices, and the mechanical characteristics of the components supporting the devices. Energy dissipation devices with stiffness and damping characteristics that are dependent on excitation frequency and/or temperature shall be modeled with characteristics consistent with (1) the deformations expected at the target displacement, and (2) a frequency equal to the inverse of the effective period.

Equation 3-11 should be used to calculate the target displacement. For velocity-dependent energy dissipation devices, the spectral acceleration in Equation 3-11 should be reduced to account for the damping afforded by the viscous dampers.

A. Displacement-Dependent Devices

Equation 3-11 should be used to calculate the target displacement. The stiffness characteristics of the energy dissipation devices should be included in the mathematical model.

B. Velocity-Dependent Devices

The target displacement of Equation 3-11 should be reduced to account for the damping added by the velocity-dependent energy dissipation devices. The calculation of the damping effect should be estimated as:

$$\beta_{eff} = \beta + \frac{\sum_j W_j}{4\pi W_k} \tag{9-36}$$

where β is the damping in the structural frame and is set equal to 0.05 unless modified in Section 2.6.1.5, W_j is work done by device j in one complete cycle corresponding to floor displacements δ_i, the summation extends over all devices j, and W_k is the maximum strain energy in the frame, determined using Equation 9-27.

The work done by device j in one complete cycle of loading may be calculated as:

$$W_j = \frac{2\pi^2}{T_s} C_j \delta_j^2 \tag{9-37}$$

where T_s is the secant fundamental period of the rehabilitated building including the stiffness of the velocity-dependent devices (if any), calculated using Equation 3-10 but replacing the effective stiffness (K_e) with the secant stiffness (K_s) at the target displacement (see Figure 9-1); C_j is the damping constant for device j; and δ_{rj} is the relative displacement between the ends of device j along the axis of device j at a roof displacement corresponding to the target displacement.

The acceptance criteria of Section 3.4.3 apply to buildings incorporating energy dissipation devices. The use of Equation 9-36 will generally capture the maximum displacement of the building. Checking for displacement-controlled actions should use deformations corresponding to the target displacement. Checking for force-controlled actions should use component actions calculated for three limit states: maximum drift, maximum velocity, and maximum acceleration. Maximum actions shall be used for design. Higher-mode effects should be explicitly evaluated.

9.3.5.2 Nonlinear Dynamic Procedure

Nonlinear Time History Analysis should be undertaken as in the requirements of Section 3.3.4.2, except as modified by this section. The mathematical model should account for both the plan and vertical spatial

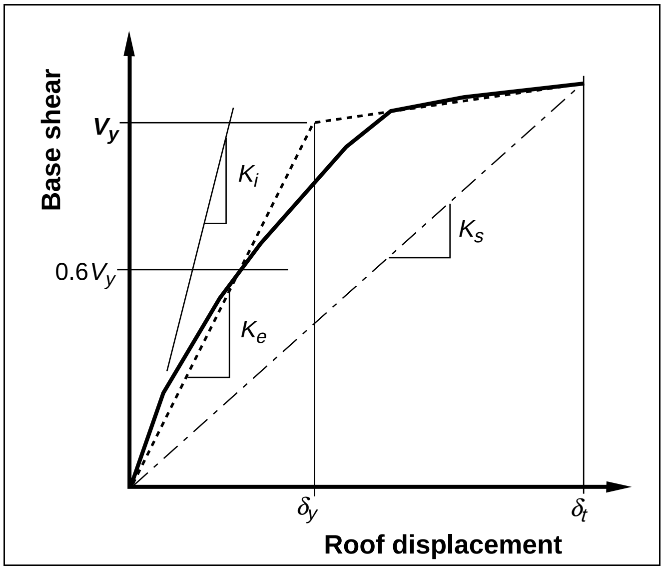


Figure 9-1 Calculation of Secant Stiffness, K_s

distribution of the energy dissipation devices in the rehabilitated building. If the energy dissipation devices are dependent on excitation frequency, operating temperature (including temperature rise due to excitation), deformation (or strain), velocity, sustained loads, and bilateral loads, such dependence should be accounted for in the analysis.

The viscous forces in velocity-dependent energy dissipation devices should be included in the calculation of design actions and deformations. Substitution of viscous effects in energy dissipation devices by global structural damping for nonlinear Time History Analysis is not permitted.

9.3.6 Detailed Systems Requirements

9.3.6.1 General

The energy dissipation system and the remainder of the lateral-force-resisting system should comply with all of the requirements of the *Guidelines*.

9.3.6.2 Operating Temperature

The force-displacement response of an energy dissipation device will generally be dependent on ambient temperature and temperature rise due to cyclic or earthquake excitation. The analysis of a rehabilitated building should account for likely variations in the force-displacement response of the energy dissipation devices to bound the seismic response of the building during the design earthquake, and develop limits for defining the acceptable response of the prototype (Section 9.3.8) and production (Section 9.3.6.6) devices.

9.3.6.3 Environmental Conditions

In addition to the requirements for vertical and lateral loads induced by wind and earthquake actions, the energy dissipation devices should be designed with consideration given to other environmental conditions, including aging effects, creep, fatigue, ambient temperature, and exposure to moisture and damaging substances.

9.3.6.4 Wind Forces

The fatigue life of energy dissipation devices, or components thereof (e.g., seals in a fluid viscous device), should be investigated and shown to be adequate for the design life of the devices. Devices subject to failure by low-cycle fatigue should resist wind forces in the linearly elastic range.

9.3.6.5 Inspection and Replacement

Access for inspection and replacement of the energy dissipation devices should be provided.

9.3.6.6 Manufacturing Quality Control

A quality control plan for manufacturing energy dissipation devices should be established by the engineer of record. This plan should include descriptions of the manufacturing processes, inspection procedures, and testing necessary to ensure quality device production.

9.3.6.7 Maintenance

The engineer of record should establish a maintenance and testing schedule for energy dissipation devices to ensure reliable response of said devices over the design life of the damper hardware. The degree of maintenance and testing should reflect the established in-service history of the devices.

9.3.7 Design and Construction Review

9.3.7.1 General

Design and construction review of all rehabilitated buildings incorporating energy dissipation devices should be performed in accordance with the requirements of Section 2.12, unless modified by the requirements of this section. Design review of the energy dissipation system and related test programs should be performed by an independent engineering review panel, including persons licensed in the appropriate disciplines, and experienced in seismic analysis including the theory and application of energy dissipation methods.

The design review should include, but should not necessarily be limited to the following:

- Preliminary design including sizing of the devices
- Prototype testing (Section 9.3.8.2)
- Final design of the rehabilitated building and supporting analyses
- Manufacturing quality control program for the energy dissipation devices

9.3.8 Required Tests of Energy Dissipation Devices

9.3.8.1 General

The force-displacement relations and damping values assumed in the design of the passive energy dissipation system should be confirmed by the following tests of a selected sample of devices prior to production of devices for construction. Alternatively, if these tests precede the design phase of a project, the results of this testing program should be used for the design.

The tests specified in this section are intended to: (1) confirm the force-displacement properties of the passive energy dissipation devices assumed for design, and (2) demonstrate the robustness of individual devices to extreme seismic excitation. These tests should not be considered as satisfying the manufacturing quality control (production) plan of Section 9.3.6.6.

The engineer of record should provide explicit acceptance criteria for the effective stiffness and damping values established by the prototype tests. These criteria should reflect the values assumed in design, account for likely variations in material properties, and provide limiting response values outside of which devices will be rejected.

The engineer of record should provide explicit acceptance criteria for the effective stiffness and damping values established by the production tests of Section 9.3.6.6. The results of the prototype tests should form the basis of the acceptance criteria for the production tests, unless an alternate basis is established by the engineer of record in the specification. Such acceptance criteria should recognize the influence of loading history on the response of individual devices by requiring production testing of devices prior to prototype testing.

The fabrication and quality control procedures used for all prototype and production devices should be identical. These procedures should be approved by the engineer of record prior to the fabrication of prototype devices.

9.3.8.2 Prototype Tests

A. General

The following prototype tests should be performed separately on two full-size devices of each type and size used in the design. If approved by the engineer of record, representative sizes of each type of device may be selected for prototype testing, rather than each type and size, provided that the fabrication and quality control procedures are identical for each type and size of devices used in the rehabilitated building.

Test specimens should not be used for construction unless approved in writing by the engineer of record.

B. Data Recording

The force-deflection relationship for each cycle of each test should be electronically recorded.

C. Sequence and Cycles of Testing

Energy dissipation devices should not form part of the gravity-load-resisting system, but may be required to support some gravity load. For the following minimum test sequence, each energy dissipation device should be loaded to simulate the gravity loads on the device as installed in the building, and the extreme ambient temperatures anticipated.

1. Each device should be loaded with the number of cycles expected in the design wind storm, but not less than 2000 fully-reversed cycles of load (displacement-dependent and viscoelastic devices) or displacement (viscous devices) at amplitudes expected in the design wind storm, at a frequency

equal to the inverse of the fundamental period of the rehabilitated building.

EXCEPTION: Devices not subject to wind-induced forces or displacements need not be subjected to these tests.

2. Each device should be loaded with 20 fully reversed cycles at the displacement in the energy dissipation device corresponding to the BSE-2, at a frequency equal to the inverse of the fundamental period of the rehabilitated building.

EXCEPTION: Energy dissipation devices may be tested by other methods than those noted above, provided that: (1) equivalency between the proposed method and cyclic testing can be demonstrated; (2) the proposed method captures the dependence of the energy dissipation device response to ambient temperature, frequency of loading, and (3) temperature rise during testing; and the proposed method is approved by the engineer of record.

D. Devices Dependent on Velocity and/or Frequency of Excitation

If the force-deformation properties of the energy dissipation devices at any displacement less than or equal to the total design displacement change by more than 15% for changes in testing frequency from 0.5 f_1 to 2.0 f_1, the preceding tests should be performed at frequencies equal to 0.5 f_1, f_1, and 2.0 f_1.

EXCEPTION: If reduced-scale prototypes are used to quantify the rate-dependent properties of energy dissipation devices, the reduced-scale prototypes should be of the same type and materials—and manufactured with the same processes and quality control procedures—as full-scale prototypes, and tested at a similitude-scaled frequency that represents the full-scale loading rates.

E. Devices Dependent on Bilateral Displacement

If the energy dissipation devices are subjected to substantial bilateral deformation, the preceding tests should be made at both zero bilateral displacement, and peak lateral displacement in the BSE-2.

EXCEPTION: If reduced-scale prototypes are used to quantify the bilateral displacement properties of the energy dissipation devices, the reduced-scale prototypes should be of the same type and materials, and manufactured with the same processes and quality control procedures, as full-scale prototypes, and tested at similitude-scaled displacements that represent the full-scale displacements.

F. Testing Similar Devices

Energy dissipation devices that are (1) of similar size, and identical materials, internal construction, and static and dynamic internal pressures (if any), and (2) fabricated with identical internal processes and manufacturing quality control procedures, that have been previously tested by an independent laboratory, in the manner described above, may not need be tested, provided that:

1. All pertinent testing data are made available to, and approved by the engineer of record.

2. The manufacturer can substantiate the similarity of the previously tested devices to the satisfaction of the engineer of record.

3. The submission of data from a previous testing program is approved in writing by the engineer of record.

9.3.8.3 Determination of Force-Displacement Characteristics

The force-displacement characteristics of an energy dissipation device should be based on the cyclic load and displacement tests of prototype devices specified in Section 9.3.8.2.

As required, the effective stiffness (k_{eff}) of an energy dissipation device with stiffness should be calculated for each cycle of deformation as follows:

$$k_{eff} = \frac{|F^-| + |F^+|}{|\Delta^-| + |\Delta^+|} \tag{9-38}$$

where forces F^+ and F^- are calculated at displacements Δ^+ and Δ^-, respectively. The effective stiffness of an energy dissipation device should be established at the test displacements in Section 9.3.8.2C.

The equivalent viscous damping of an energy dissipation device (β_{eff}) exhibiting stiffness should be calculated for each cycle of deformation as:

$$\beta_{eff} = \frac{1}{2\pi} \frac{W_D}{k_{eff} \Delta_{ave}^2} \quad (9\text{-}39)$$

where k_{eff} is established in Equation 9-38, and W_D is the area enclosed by one complete cycle of the force-displacement response for a single energy dissipation device at a prototype test displacement (Δ_{ave}) equal to the average of the absolute values of displacements Δ^+ and Δ^-.

9.3.8.4 System Adequacy

The performance of a prototype device may be assessed as adequate if all of the following conditions are satisfied:

1. The force-displacement curves for the tests in Section 9.3.8.2C have nonnegative incremental force-carrying capacities.

   EXCEPTION: Energy dissipation devices that exhibit velocity-dependent behavior need not comply with this requirement.

2. Within each test of Section 9.3.8.2C, the effective stiffness (k_{eff}) of a prototype energy dissipation device for any one cycle does not differ by more than plus or minus 15% from the average effective stiffness as calculated from all cycles in that test.

   EXCEPTIONS: (1) The 15% limit may be increased by the engineer of record in the specification, provided that the increased limit has been demonstrated by analysis to not have a deleterious effect on the response of the rehabilitated building. (2) Fluid viscous energy dissipation devices, and other devices that do not have effective stiffness, need not comply with this requirement.

3. Within each test of Section 9.3.8.2C, the maximum force and minimum force at zero displacement for a prototype device for any one cycle does not differ by more than plus or minus 15% from the average maximum and minimum forces as calculated from all cycles in that test.

   EXCEPTION: The 15% limit may be increased by the engineer of record in the specification, provided that the increased limit has been demonstrated by analysis to not have a deleterious effect on the response of the rehabilitated building.

4. Within each test of Section 9.3.8.2C, the area of the hysteresis loop (W_D) of a prototype energy dissipation device for any one cycle does not differ by more than plus or minus 15% from the average area of the hysteresis curve as calculated from all cycles in that test.

   EXCEPTION: The 15% limit may be increased by the engineer of record in the specification, provided that the increased limit has been demonstrated by analysis to not have a deleterious effect on the response of the rehabilitated building.

5. For displacement-dependent devices, the average effective stiffness, average maximum and minimum force at zero displacement, and average area of the hysteresis loop (W_D), calculated for each test in the sequence described in Section 9.3.8.2C, shall fall within the limits set by the engineer-of-record in the specification. The area of the hysteresis loop at the end of cyclic testing should not differ by more than plus or minus 15% from the average area of the 20 test cycles.

6. For velocity-dependent devices, the average maximum and minimum force at zero displacement, effective stiffness (for viscoelastic devices only), and average area of the hysteresis loop (W_D), calculated for each test in the sequence described in Section 9.3.8.2C, shall fall within the limits set by the engineer of record in the specification.

9.4 Other Response Control Systems

Response control strategies other than base isolation (Section 9.2) and passive energy dissipation (Section 9.3) systems have been proposed. Dynamic vibration absorption and active control systems are two such response control strategies. Although both dynamic vibration absorption and active control systems have been implemented to control the wind-induced vibration of buildings, the technology is not sufficiently mature, and the necessary hardware is not sufficiently robust, to warrant the preparation of general guidelines for the implementation of other response control systems. However, *Commentary* Section C9.4 provides a more detailed discussion of two other systems: dynamic vibration absorbers and active control systems.

The analysis and design of other response control systems should be reviewed by an independent engineering review panel per the requirements of Section 9.3.7. This review panel should include persons expert in the theory and application of the response control strategies being considered, and should be impaneled by the owner prior to the development of the preliminary design.

9.5 Definitions

BSE-1: Basic Safety Earthquake-1, which is the lesser of the ground shaking at a site for a 10%/50 year earthquake or two thirds of the MCE earthquake at the site.

BSE-2: Basic Safety Earthquake-2, which is the ground shaking at a site for an MCE earthquake.

Design displacement: The design earthquake displacement of an isolation or energy dissipation system, or elements thereof, excluding additional displacement due to actual and accidental torsion.

Design earthquake: A user-specified earthquake for the design of an isolated building, having ground shaking criteria described in Chapter 2.

Displacement-dependent energy dissipation devices: Devices having mechanical properties such that the force in the device is related to the relative displacement in the device.

Displacement restraint system: Collection of structural components and elements that limit lateral displacement of seismically-isolated buildings during the BSE-2.

Effective damping: The value of equivalent viscous damping corresponding to the energy dissipated by the building, or element thereof, during a cycle of response.

Effective stiffness: The value of the lateral force in the building, or an element thereof, divided by the corresponding lateral displacement.

Energy dissipation device (EDD): Non-gravity-load-supporting element designed to dissipate energy in a stable manner during repeated cycles of earthquake demand.

Energy dissipation system (EDS): Complete collection of all energy dissipation devices, their supporting framing, and connections.

Isolation interface: The boundary between the upper portion of the structure (superstructure), which is isolated, and the lower portion of the structure, which moves rigidly with the ground.

Isolation system: The collection of structural elements that includes all individual isolator units, all structural elements that transfer force between elements of the isolation system, and all connections to other structural elements. The isolation system also includes the wind-restraint system, if such a system is used to meet the design requirements of this section.

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

Maximum displacement: The maximum earthquake displacement of an isolation or energy dissipation system, or elements thereof, excluding additional displacement due to actual or accidental torsion.

Tie-down system: The collection of structural connections, components, and elements that provide restraint against uplift of the structure above the isolation system.

Total design displacement: The BSE-1 displacement of an isolation or energy dissipation system, or elements thereof, including additional displacement due to actual and accidental torsion.

Total maximum displacement: The maximum earthquake displacement of an isolation or energy dissipation system, or elements thereof, including additional displacement due to actual and accidental torsion.

Velocity-dependent energy dissipation devices: Devices having mechanical characteristics such that the force in the device is dependent on the relative velocity in the device.

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.

9.6 Symbols

This list may not contain symbols defined at their first use if not used thereafter.

B_{D1}	Numerical coefficient taken equal to the value of β_1, as set forth in Table 2-15, at effective damping equal to the value of β_D
B_{M1}	Numerical coefficient taken equal to the value of β_1, as set forth in Table 2-15, at effective damping equal to the value of β_M
C or C_j	Damping coefficient
CF_i	State combination factors for use with velocity-dependent energy dissipation devices
D	Target spectral displacement
D	Displacement of an energy dissipation unit
D_{ave}	Average displacement of an energy dissipation unit, equal to $(\lvert D^+\rvert + \lvert D^-\rvert)/2$
D^-	Maximum negative displacement of an energy dissipation unit
D^+	Maximum positive displacement of an energy dissipation unit
$\dot{D}$	Relative velocity of an energy dissipation unit
D_D	Design displacement, in in. (mm), at the center of rigidity of the isolation system in the direction under consideration, as prescribed by Equation 9-2
D'_D	BSE-1 displacement, in in. (mm), at the center of rigidity of the isolation system in the direction under consideration, as prescribed by Equation 9-10
D_M	Maximum displacement, in in. (mm), at the center of rigidity of the isolation system in the direction under consideration, as prescribed by Equation 9-4
D'_M	Maximum displacement, in in. (mm), at the center of rigidity of the isolation system in the direction under consideration, as prescribed by Equation 9-11
D_{TD}	Total design displacement, in in. (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 specified by Equation 9-6
D_{TM}	Total maximum displacement, in in. (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 specified by Equation 9-7
E_{Loop}	Energy dissipated, in kip-inches (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 force-deflection curve
F	Force in an energy dissipation unit
F^-	Negative force, in k, in an isolator or energy dissipation unit during a single cycle of prototype testing at a displacement amplitude of Δ^-
F^+	Positive force, in k, in an isolator or energy dissipation unit during a single cycle of prototype testing at a displacement amplitude of Δ^+
K_{Dmax}	Maximum effective stiffness, in k/in., of the isolation system at the design displacement in the horizontal direction under consideration, as prescribed by Equation 9-14
K_{Dmin}	Minimum effective stiffness, in k/in., of the isolation system at the design displacement in the horizontal direction under consideration, as prescribed by Equation 9-15
K'	Storage stiffness
K''	Loss stiffness

K_{Mmax} — Maximum effective stiffness, in k/in., of the isolation system at the maximum displacement in the horizontal direction under consideration, as prescribed by Equation 9-16

K_{Mmin} — Minimum effective stiffness, in k/in., of the isolation system at the maximum displacement in the horizontal direction under consideration, as prescribed by Equation 9-17

S_{D1} — One-second, 5%-damped spectral acceleration for the design earthquake, as set forth in Chapter 2

S_{DS} — Short-period, 5%-damped spectral acceleration for the design earthquake, as set forth in Chapter 2

S_{M1} — One-second, 5%-damped spectral acceleration, as set forth in Chapter 2 for the BSE-2

S_{MS} — Short-period, 5%-damped spectral acceleration, as set forth in Chapter 2 for the BSE-2

T_D — Effective period, in seconds, of the seismic-isolated structure at the design displacement in the direction under consideration, as prescribed by Equation 9-3

T_e — Effective fundamental-mode period, in seconds, of the building in the direction under consideration

T_M — Effective period, in seconds, of the seismic-isolated structure at the maximum displacement in the direction under consideration, as prescribed by Equation 9-5

T_s — Secant fundamental period of a rehabilitated building calculated using Equation 3-10 but replacing the effective stiffness (K_e) with the secant stiffness (K_s) at the target displacement

V^* — Modified equivalent base shear

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 Equation 9-8

V_s — The total lateral seismic design force or shear on elements above the isolation system, as prescribed by Section 9.2.4.4B

V_t — Total base shear determined by Time-History Analysis

W — The total seismic dead load. For design of the isolation system, W is the total seismic dead load weight of the structure above the isolation interface

W_D — Energy dissipated, in in.-k, in a building or element thereof during a full cycle of displacement

b — The shortest plan dimension of the rehabilitated building, in ft (mm), measured perpendicular to d

d — The longest plan dimension of the rehabilitated building, in ft (mm)

e — Actual eccentricity, ft (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 (mm), taken as 5% of the maximum building dimension perpendicular to the direction of force under consideration

f_1 — Fundamental frequency of the building

g — Acceleration of gravity (386.1 in/sec.^{2}, or 9,800 mm/sec.^{2} for SI units)

k_{eff} — Effective stiffness of an isolator unit, as prescribed by Equation 9-12, or an energy dissipation unit, as prescribed by Equation 9-38

m — Mass (k-sec^2/in.)

q — Coefficient, less than one, equal to the ratio of actual hysteresis loop area to idealized bilinear hysteresis loop area

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

Δ_{ave} — Average displacement of an energy dissipation unit during a cycle of prototype testing, equal to $(|\Delta^+| + |\Delta^-|)/2$

Δ^+ — Positive displacement amplitude, in in. (mm), of an isolator or energy dissipation unit during a cycle of prototype testing

Δ^- Negative displacement amplitude, in in. (mm), of an isolator or energy dissipation unit during a cycle of prototype testing

ΣE_D Total energy dissipated, in in.-k, in the isolation system during a full cycle of response at the design displacement, D_D

ΣE_M Total energy dissipated, in in.-k, in the isolation system during a full cycle of response at the maximum displacement, D_M

$\Sigma|F_D^+|_{max}$ Sum, for all isolator units, of the maximum absolute value of force, k, at a positive displacement equal to D_D

$\Sigma|F_D^+|_{min}$ Sum, for all isolator units, of the minimum absolute value of force, k, at a positive displacement equal to D_D

$\Sigma|F_D^-|_{max}$ Sum, for all isolator units, of the maximum absolute value of force, k, at a negative displacement equal to D_D

$\Sigma|F_D^-|_{min}$ Sum, for all isolator units, of the minimum absolute value of force, k, at a negative displacement equal to D_D

$\Sigma|F_M^+|_{max}$ Sum, for all isolator units, of the maximum absolute value of force, k, at a positive displacement equal to D_M

$\Sigma|F_M^+|_{min}$ Sum, for all isolator units, of the minimum absolute value of force, k, at a positive displacement equal to D_M

$\Sigma|F_M^-|_{max}$ Sum, for all isolator units, of the maximum absolute value of force, k, at a negative displacement equal to D_M

$\Sigma|F_M^-|_{min}$ Sum, for all isolator units, of the minimum absolute value of force, k, at a negative displacement equal to D_M

β Damping inherent in the building frame (typically equal to 0.05)

β_b Equivalent viscous damping of a bilinear system

β_{eff} Effective damping of isolator unit, as prescribed by Equation 9-13, or an energy dissipation unit, as prescribed by Equation 9-39; also used for the effective damping of the building, as prescribed by Equations 9-26, 9-30, and 9-36

β_D Effective damping of the isolation system at the design displacement, as prescribed by Equation 9-18

β_M Effective damping of the isolation system at the maximum displacement, as prescribed by Equation 9-19

δ_i Floor displacement

θ_j Angle of inclination of energy dissipation device

ϕ_i Modal displacement of floor i

ϕ_{rj} Relative modal displacement in horizontal direction of energy dissipation device j

ω_1 $2\pi f_1$

9.7 References

BSSC, 1995, *NEHRP Recommended Provisions for Seismic Regulations for New Buildings, 1994 Edition, Part 1: Provisions and Part 2: Commentary*, prepared by the Building Seismic Safety Council for the Federal Emergency Management Agency (Report Nos. FEMA 222A and 223A), Washington, D.C.

BSSC, 1997, *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and other Structures, 1997 Edition, Part 1: Provisions and Part 2: Commentary*, prepared by the Building Seismic Safety Council for the Federal Emergency Management Agency (Report Nos. FEMA 302 and 303), Washington, D.C.

Secretary of the Interior, 1993, *Standards and Guidelines for Archaeology and Historic Preservation*, published in the *Federal Register*, Vol. 48, No. 190, pp. 44716–44742.

10. Simplified Rehabilitation

10.1 Scope

This chapter presents the Simplified Rehabilitation Method, which is intended primarily for use on a selected group of simple buildings being rehabilitated to the Life Safety Performance Level for the level of ground motion specified in FEMA 178, *NEHRP Handbook for the Seismic Evaluation of Existing Buildings* (BSSC, 1992a). In an area of low or moderate seismicity, designing for this level of ground motion may not be sufficient to provide Life Safety Performance if a large infrequent earthquake occurs.

The technique described in this chapter is one of the two rehabilitation methods defined in Chapter 2. It is to be used only by a design professional, and only in a manner consistent with the *Guidelines*. Consideration must be given to all aspects of the rehabilitation process, including the development of appropriate as-built information, proper design of rehabilitation techniques, and specification of appropriate levels of quality assurance. Systematic Rehabilitation is the other rehabilitation method defined in Chapter 2.

The term "Simplified Rehabilitation" is intended to reflect a level of analysis and design that (1) is appropriate for small, regular buildings, and buildings that do not require advanced analytical procedures, and (2) does not achieve the Basic Safety Objective (BSO).

FEMA 178 (BSSC, 1992a), the *NEHRP Handbook for the Seismic Evaluation of Existing Buildings*, a nationally applicable evaluation method, is the basis for the Simplified Rehabilitation Method. FEMA 178 is based on the historic behavior of buildings in past earthquakes and the success of current code provisions in achieving the Life Safety Performance Level. It is organized around a set of common construction styles called model buildings. The performance of certain common building types that meet specific limitations on height and regularity can be substantially improved by simply eliminating all of the deficiencies found using FEMA 178. See Section C10.1 in the *Commentary* for further information on FEMA 178 and other introductory comments. FEMA 178 is currently under revision (October, 1997) and the revised version will be available soon. These *Guidelines* refer frequently to FEMA 178 as a pointer to the FEMA 178 references.

Since the preliminary version of FEMA 178 was completed in the late 1980s, new information has become available, which will be added to FEMA 178 in the updated edition of the document now underway. This information has been included in the Simplified Rehabilitation Method, presented as amendments to FEMA 178 (BSSC, 1992a), and includes additional Model Building Types and eight new evaluation statements for new potential deficiencies. They are presented in the same format and style as used in FEMA 178. The set of common Model Building Types has been expanded to separate those buildings with stiff and flexible diaphragms, and to account for the unique behavior of multistory, multi-unit, wood-frame structures. While near-fault effects are also being proposed to amend FEMA 178, they are not expected to affect the buildings eligible for Simplified Rehabilitation and therefore need not be considered.

The evaluation statements and procedures contained in FEMA 178 apply best to low-rise and, in some cases, mid-rise buildings of regular configuration and well-defined building type. Table 10-1 identifies those buildings for which the Simplified Rehabilitation Method can be used to achieve the Life Safety Performance Level for ground motions specified in FEMA 178 (BSSC, 1992a). It is required, however, that the building deficiencies be corrected by strengthening and/or modifying the existing components of the building using the same basic style of construction. Buildings that have configuration irregularities, as defined in the *NEHRP Recommended Provisions for Regulations for New Buildings* (BSSC, 1995), may use this Simplified Rehabilitation Method to achieve the Life Safety Performance Level only if the resulting rehabilitation work eliminates all significant vertical and horizontal irregularities and results in a building with a complete seismic lateral-force-resisting load path.

The Simplified Rehabilitation Method may be used to achieve Limited Rehabilitation Objectives for any building not listed in Table 10-1. (Note that Table 10-1, the remaining Tables 10-2 to 10-22, and Figure 10-1 are at the end of this chapter.)

The Simplified Rehabilitation Method may yield a more conservative result than the Systematic Method. This is due to the variety of simplifying assumptions. Because of the small size and simplicity of the buildings that are eligible for the Simplified Method of achieving the Life Safety Performance Level, the economic consequences of this conservatism are likely to be insignificant. It must be understood, however, that a simple comparison of the design-based shear in FEMA 178 with Chapter 3 of the *Guidelines* will lead to the opposite conclusion. The equivalent lateral forces used in these two documents have entirely different definitions and bases. The FEMA 178 (BSSC, 1992a) values, which are based on the traditional techniques used in building codes, have been developed on a different basis than in Chapter 3 of the *Guidelines*, and have been taken from the 1988 *NEHRP Provisions*. The Chapter 3 values calculated from a "pseudo lateral load," are defined for a component-based analysis and do not include the same reduction factors. As shown in Figure 10-1, while the base and story shear values may vary by approximately six times, the ratios of demand/capacity vary only slightly.

Implementing a rehabilitation scheme that mitigates all of a building's FEMA 178 (BSSC, 1992a) deficiencies using the Simplified Rehabilitation Method does not in and of itself achieve the Basic Safety Objective or any Enhanced Rehabilitation Objective as defined in Chapter 2, since the rehabilitated building may not meet the Collapse Prevention Performance Level for BSE-2. If the goal is to attain the Basic Safety Objective as described in Chapter 2 or other Enhanced Rehabilitation Objectives, this can be accomplished by using the Systematic Rehabilitation Method defined in Chapter 2.

10.2 Procedural Steps

The application of the Simplified Rehabilitation Method first requires a complete FEMA 178 (BSSC, 1992a) evaluation of a building, which results in a list of deficiencies. These deficiencies are then ranked, and common and simple rehabilitation procedures are applied to correct them. Once a full rehabilitation scheme has been devised, the building is reevaluated using FEMA 178 to verify that it fully meets the requirements. A more complete statement of this procedure follows. The procedures are applicable only to buildings that meet the qualification criteria shown in Table 10-1.

1. Identify the model building type. Each is described in Table 10-2 and in more detail in FEMA 178 (BSSC, 1992a). The building must be one of the common building types and satisfy the criteria described in Table 10-1.

2. Identify and rank all potential deficiencies for the building from Tables 10-3 through 10-21. The items in these tables are ordered roughly from highest priority at the top to lowest at the bottom, though this can vary widely in individual cases. Develop as-built information as required in *Guidelines* Section 2.7. Use the procedures in FEMA 178—and also those listed in *Guidelines* Section 10.4 for the eight new potential deficiencies—in order to evaluate fully each potential deficiency and develop a list of actual deficiencies in priority order for correction. If necessary, refer to Section C10.5 of the *Commentary* for a complete list of FEMA 178 deficiencies and their relationship to the deficiency list used here. Table 10-22 provides a cross-reference between all FEMA 178 (BSSC, 1992a) deficiencies and those of this chapter.

3. Develop strengthening details to mitigate the deficiencies using the same basic style and materials of construction. Refer to Section 10.3 and the *Commentary* for rehabilitation strategies associated with each identified deficiency. In most cases, the resulting rehabilitated building must be one of the Model Building Types. For example, adding concrete shear walls to concrete shear wall buildings or adding a complete system of concrete shear walls to a concrete frame building meets this requirement. Some exceptions include using steel bracing to strengthen wood or URM construction. For large buildings, it is advisable to explore several rehabilitation strategies and compare alternative ways of eliminating deficiencies.

4. Design the proposed rehabilitation based on the FEMA 178 (BSSC, 1992a) criteria, including its Appendix C, such that all deficiencies are eliminated.

5. Once rehabilitation techniques have been developed for all deficiencies, perform a complete evaluation of the building in its proposed rehabilitated state, following the FEMA 178 (BSSC, 1992a) procedures. This step should confirm that the strengthening of any one element or system has not merely shifted the deficiency to another.

6. To achieve the BSO, consider the rehabilitated structure's potential performance using the

Systematic Rehabilitation Method. Determine whether the total strength of the building is sufficient, and judge whether the building can experience the predicted maximum displacement without partial or complete collapse.

7. Identify and develop strengthening details for the architectural, mechanical, and electrical components. Refer to the procedures in Chapter 11 for the evaluation and rehabilitation of nonstructural elements related to the Life Safety Performance Level, given the BSE-1 earthquake.

8. Develop the needed construction documents, including drawings and specifications, and include an appropriate quality assurance program as defined in Chapter 2. If only partial rehabilitation is intended, it is recommended that the deficiencies be corrected in priority order and in a way that will facilitate fulfillment of the requirements of a higher objective at a later date. Care must be taken to ensure that a partial rehabilitation effort does not make the building's overall performance worse, such as by unintentionally channeling failure to a more critical element.

10.3 Suggested Corrective Measures for Deficiencies

Tables 10-3 to 10-21 list the potential deficiencies for the various Model Building Types. Each of these may be shown to be a deficiency that needs correction during a rehabilitation effort. (See *Commentary* Section C10.5 for a complete list of evaluation statements for identifying potential deficiencies, both those in FEMA 178 (BSSC, 1992a) and the Amendments to FEMA 178 in these *Guidelines*, Section 10.4. The following sections describe suggested corrective measures for each deficiency. They are organized into deficiency groups similar to those used in FEMA 178, and are intended to assist the thinking of the design professional. Other appropriate solutions may be used. The *Commentary* provides further discussion of the ranking of the deficiencies.

10.3.1 Building Systems

10.3.1.1 Load Path

Load path discontinuities can be mitigated by adding elements to complete the load path. This may require adding new well-founded shear walls or frames to fill in the gaps in existing shear walls or frames that are not carried continuously all the way down to the foundation. Alternatively, it may require the addition of elements throughout the building to pick up loads from diaphragms that have no path into existing vertical elements. (FEMA 178 [BSSC, 1992a], Section 3.1.)

10.3.1.2 Redundancy

The most prudent rehabilitation strategy for a building without redundancy is to add new lateral-force-resisting elements in locations where the failure of a single element will cause an instability in the building. The added lateral-force-resisting elements should be of the same stiffness as the elements they are supplementing. It is not generally satisfactory just to strengthen a nonredundant element (such as by adding cover plates to a slender brace), because its failure would still result in an instability. (FEMA 178 [BSSC, 1992a], Section 3.2.)

10.3.1.3 Vertical Irregularities

New vertical lateral-force-resisting elements can be provided to eliminate the vertical irregularity. For weak stories, soft stories, and vertical discontinuities, new elements of the same type can be added as needed. Mass and geometric discontinuities must be evaluated and strengthened based on Systematic Rehabilitation, if required by Chapter 2. (FEMA 178 [BSSC, 1992a], Sections 3.3.1 through 3.3.5.)

10.3.1.4 Plan Irregularities

The effects of plan irregularities that create torsion can be eliminated with the addition of lateral-force-resisting bracing elements that will support all major diaphragm segments in a balanced manner. While it is possible in some cases to allow the irregularity to remain and instead strengthen those structural elements that are overstressed by its existence, this may require substantial additional analysis, does not directly address the problem, and requires use of the Systematic Rehabilitation Method. (FEMA 178 [BSSC, 1992a], Section 3.3.6.)

10.3.1.5 Adjacent Buildings

Stiffening elements (typically braced frames or shear walls) can be added to one or both buildings to reduce the expected drifts to acceptable levels. With separate structures in a single building complex, it may be possible to tie them together structurally to force them to respond as a single structure. The relative stiffnesses of each and the resulting force interactions must be determined to ensure that additional deficiencies are not created. Pounding can also be eliminated by demolishing a portion of one building to increase the separation. (FEMA 178 [BSSC, 1992a], Section 3.4.)

10.3.1.6 Lateral Load Path at Pile Caps

Typically, deficiencies in the load path at the pile caps are not a life safety concern. However, if the design professional has determined that there is strong possibility of a life safety hazard due to this deficiency, piles and pile caps may be modified, supplemented, repaired, or in the most severe condition, replaced in their entirety. Alternatively, the building system may be rehabilitated such that the pile caps are protected.

10.3.1.7 Deflection Compatibility

Vertical lateral-force-resisting elements can be added to decrease the drift demands on the columns, or the ductility of the columns can be increased. Jacketing the columns with steel or concrete is one approach to increase their ductility.

10.3.2 Moment Frames

10.3.2.1 Steel Moment Frames

A. Drift

The most direct mitigation approach is to add properly placed and distributed stiffening elements—such as new moment frames, braced frames, or shear walls—that can reduce the inter-story drifts to acceptable levels. Alternatively, the addition of energy dissipation devices to the system may reduce the drift, though these are outside the scope of Simplified Rehabilitation. (FEMA 178 [BSSC, 1992a], Section 4.2.1.)

B. Frames

Noncompact members can be eliminated by adding appropriate steel plates. Eliminating or properly reinforcing large member penetrations will develop the demanded strength and deformations. Lateral bracing in the form of new steel elements can be added to reduce member unbraced lengths to within the limits prescribed. Stiffening elements (e.g., braced frames, shear walls, or additional moment frames) can be added throughout the building to reduce the expected frame demands. (FEMA 178 [BSSC, 1992a], Sections 4.2.2, 4.2.3, and 4.2.9.)

C. Strong Column-Weak Beam

Steel plates can be added to increase the strength of the steel columns to beyond that of the beams, to eliminate this issue. Stiffening elements (e.g., braced frames, shear walls, or additional moment frames) can be added throughout the building to reduce the expected frame demands. (FEMA 178 [BSSC, 1992a], Section 4.2.8.)

D. Connections

Adding a stiffer lateral-force-resisting system (e.g., braced frames or shear walls) can reduce the expected rotation demands. Connections can be modified by adding flange cover plates, vertical ribs, haunches, or brackets, or removing beam flange material to initiate yielding away from the connection location (e.g., via a pattern of drilled holes or the cutting out of flange material). Partial penetration splices, which may become more vulnerable for conditions where the beam-column connections are modified to be more ductile, can be modified by adding plates and/or welds. Adding continuity plates alone is not likely to enhance the connection performance significantly. (FEMA 178 [BSSC, 1992a], Sections 4.2.4, 4.2.5, 4.2.6, and 4.2.7.)

Moment-resisting connection capacity can be increased by adding cover plates or haunches, or using other techniques as stipulated in the SAC *Interim Guidelines*, FEMA 267 (SAC, 1995).

10.3.2.2 Concrete Moment Frames

A. Frame and Nonductile Detail Concerns

Adding properly placed and distributed stiffening elements such as shear walls will fully supplement the moment frame system with a new lateral-force-resisting system. For eccentric joints, columns and/or beams may be jacketed to reduce the effective eccentricity. Jackets may also be provided for shear-critical columns.

It must be verified that this new system sufficiently reduces the frame shears and inter-story drifts to acceptable levels. (FEMA 178 [BSSC, 1992a], Sections 4.3.1–4.3.15.)

B. Precast Moment Frames

Precast concrete frames without shear walls may not be addressed under the Simplified Rehabilitation Method (see Table 10-1). Where shear walls are present, the precast connections must be strengthened sufficiently to meet the FEMA 178 (BSSC, 1992a) requirements.

The development of a competent load path is extremely critical in these buildings. If the connections have sufficient strength so that yielding will first occur in the members rather than in the connections, the building should be evaluated as a shear wall system (Type C2). (FEMA 178 [BSSC, 1992a] Section 4.4.1.)

10.3.2.3 Frames Not Part of the Lateral-Force-Resisting System

A. Complete Frames

Complete frames, of steel or concrete, form a complete vertical-load-carrying system.

Incomplete frames are essentially bearing wall systems. The wall must be strengthened to resist the combined gravity/seismic loads or new columns added to complete the gravity load path. (FEMA 178 [BSSC, 1992a], Section 4.5.1.)

B. Short Captive Columns

Columns may be jacketed with steel or concrete such that they can resist the expected forces and drifts. Alternatively, the expected story drifts can be reduced throughout the building by infilling openings or adding shear walls. (Section 10.4.2.2.)

10.3.3 Shear Walls

10.3.3.1 Cast-in-Place Concrete Shear Walls

A. Shearing Stress

New shear walls can be provided and/or the existing walls can be strengthened to satisfy seismic demand criteria. New and strengthened walls must form a complete, balanced, and properly detailed lateral-force-resisting system for the building. Special care is needed to ensure that the connection of the new walls to the existing diaphragm is appropriate and of sufficient strength such that yielding will first occur in the wall. All shear walls must have sufficient shear and overturning resistance to meet the FEMA 178 load criteria. (FEMA 178 [BSSC, 1992a], Section 5.1.1.)

B. Overturning

Lengthening or adding shear walls can reduce overturning demands; increasing the length of footings will capture additional building dead load. (FEMA 178 [BSSC, 1992a], Section 5.1.2.)

C. Coupling Beams

To eliminate the need to rely on the coupling beam, the walls may be strengthened as required. The beam should be jacketed only as a means of controlling debris. If possible, the opening that defines the coupling beam should be infilled. (FEMA 178 [BSSC, 1992a], Section 5.1.3.)

D. Boundary Component Detailing

Splices may be improved by welding bars together after exposing them. The shear transfer mechanism can be improved by adding steel studs and jacketing the boundary components. (FEMA 178 [BSSC, 1992a], Sections 5.1.4 through 5.1.6.)

E. Wall Reinforcement

The shear walls can be strengthened by infilling openings, or by thickening the walls (see FEMA 172 [BSSC, 1992b], Section 3.2.1.2). (FEMA 178 [BSSC, 1992a], Sections 5.1.7 and 5.1.8.)

10.3.3.2 Precast Concrete Shear Walls

A. Panel-to-Panel Connections

Appropriate Simplified Rehabilitation solutions are outlined in FEMA 172, Section 3.2.2.3. (FEMA 178 [BSSC, 1992a], Section 5.2.1.)

Inter-panel connections with inadequate capacity can be strengthened by adding steel plates across the joint, or by providing a continuous wall by exposing the reinforcing steel in the adjacent units, providing ties between the panels and patching with concrete. Providing steel plates across the joint is typically the most cost-effective approach, although care must be taken to ensure adequate anchor bolt capacity by providing adequate edge distances (see FEMA 172, Section 3.2.2).

B. Wall Openings

Infilling openings or adding shear walls in the plane of the open bays can reduce demand on the connections and eliminate frame action. (FEMA 178 [BSSC, 1992a], Section 5.2.2.)

C. Collectors

Upgrading the concrete section and/or the connections (e.g., exposing the existing connection, adding confinement ties, increasing embedment) can increase strength and/or ductility. Alternative load paths for lateral forces can be provided, and shear walls added to reduce demand on the existing collectors. (FEMA 178 [BSSC, 1992a], Section 5.2.3.)

10.3.3.3 Masonry Shear Walls

A. Reinforcing in Masonry Walls

Nondestructive methods should be used to locate reinforcement, and selective demolition used if necessary to determine the size and spacing of the reinforcing. If it cannot be verified that the wall is reinforced in accordance with the minimum requirements, then the wall should be assumed to be unreinforced, and therefore must be supplemented with new walls, or the procedures for unreinforced masonry should be followed. (FEMA 178 [BSSC, 1992a], Section 5.3.2.)

B. Shearing Stress

To meet the lateral force requirements of FEMA 178 (BSSC, 1992a), new walls can be provided, or the existing walls strengthened as needed. New and strengthened walls must form a complete, balanced, and properly detailed lateral-force-resisting system for the building. Special care is needed to ensure that the connection of the new walls to the existing diaphragm is appropriate and of sufficient strength to deliver the actual lateral loads or force yielding in the wall. All shear walls must have sufficient shear and overturning resistance.

C. Reinforcing at Openings

The presence and location of reinforcing steel at openings may be established using nondestructive or destructive methods at selected locations to verify the size and location of the reinforcing, or using both methods. Reinforcing must be provided at all openings as required to meet the FEMA 178 criteria. Steel plate may be bolted to the surface of the section as long as the bolts are sufficient to yield the steel plate. (FEMA 178 [BSSC, 1992a], Section 5.3.3.)

D. Unreinforced Masonry Shear Walls

Openings in the lateral-force-resisting walls should be infilled as needed to meet the FEMA 178 (BSSC, 1992a) stress check. If supplemental strengthening is required, it should be designed using the Systematic Rehabilitation Method as defined in Chapter 2. Walls that do not meet the masonry lay-up requirements should not be considered as lateral-force-resisting elements and shall be specially supported for out-of-plane loads. (FEMA 178 [BSSC, 1992a], Sections 5.4.1, 5.4.2.)

E. Proportions of Solid Walls

Walls with insufficient thickness should be strengthened either by increasing the thickness of the wall or by adding a well-detailed strong back system. The thickened wall must be detailed in a manner that fully interconnects the wall over its full height. The strong back system must be designed for strength, connected to the structure in a manner that: (1) develops the full yield strength of the strong back, and (2) connects to the diaphragm in a manner that distributes the load into the diaphragm and has sufficient stiffness to ensure that the elements will perform in a compatible and acceptable manner. The stiffness of the bracing should limit the out-of-plane deflections to acceptable levels such as L/600 to L/900 (FEMA 178 [BSSC, 1992a], Sections 5.5.1, 5.5.2.)

F. Infill Walls

The partial infill wall should be isolated from the boundary columns to avoid a “short column” effect, except when it can be shown that the column is adequate. In sizing the gap between the wall and the columns, the anticipated inter-story drift must be considered. The wall must be positively restrained against out-of-plane failure by either bracing the top of the wall, or installing vertical girts. These bracing elements must not violate the isolation of the frame from the infill. (FEMA 178 [BSSC, 1992a], Sections 5.5.3, 4.1.1.)

10.3.3.4 Shear Walls in Wood Frame Buildings

A. Shear Stress

Walls may be added or existing openings filled. Alternatively, the existing walls and connections can be strengthened. The walls should be distributed across the building in a balanced manner to reduce the shear stress for each wall. Replacing heavy materials such as tile roofing with lighter materials will also reduce shear stress. (FEMA 178 [BSSC, 1992a], Section 5.6.1.)

B. Openings

Local shear transfer stresses can be reduced by distributing the forces from the diaphragm. Chords and/or collector members can be provided to collect and distribute shear from the diaphragm to the shear wall or bracing (see FEMA 172, Figure 3.7.1.3). Alternatively, the opening can be closed off by adding a new wall with plywood sheathing. (FEMA 178 [BSSC, 1992a], Section 5.6.2.)

C. Wall Detailing

If the walls are not bolted to the foundation or the bolting is inadequate, bolts can be installed through the sill plates at regular intervals (see FEMA 172 [BSSC, 1992b], Figure 3.8.1.2a). If the crawl space is not deep enough for vertical holes to be drilled through the sill plate, the installation of connection plates or angles may be a more practical alternative (see FEMA 172, Figure 3.8.1.2b). Sheathing and additional nailing can be added where walls lack proper nailing or connections. Where the existing connections are inadequate, adding clips or straps will deliver lateral loads to the walls and to the foundation sill plate. (FEMA 178 [BSSC, 1992a], Section 5.6.3.)

D. Cripple Walls

Where bracing is inadequate, new plywood sheathing can be added to the cripple wall studs. The top edge of the plywood is nailed to the floor framing and the bottom edge is nailed into the sill plate (see FEMA 172, Figure 3.8.1.3). Verify that the cripple wall does not change height along its length (stepped top of foundation). If it does, the shorter portion of the cripple wall will carry the majority of the shear and significant torsion will occur in the foundation. Added plywood sheathing must have adequate strength and stiffness to reduce torsion to an acceptable level. Also, it should be verified that the sill plate is properly anchored to the foundation. If anchor bolts are lacking or insufficient, additional anchor bolts should be installed. Blocking and/or framing clips may be needed to connect the cripple wall bracing to the floor diaphragm or the sill plate. (FEMA 178 [BSSC, 1992a], Section 5.6.4.)

E. Narrow Wood Shear Walls

Where narrow shear walls lack capacity, they should be replaced with shear walls with a height-to-width aspect ratio of two to one or less. These replacement walls must have sufficient strength, including being adequately connected to the diaphragm and sufficiently anchored to the foundation for shear and overturning forces. (*Guidelines* Section 10.4.3.1.)

F. Stucco Shear Walls

For strengthening or repair, the stucco should be removed, a plywood shear wall added, and new stucco applied. The plywood should be the manufacturer’s recommended thickness for the installation of stucco. The new stucco should be installed in accordance with building code requirements for waterproofing. Walls should be sufficiently anchored to the diaphragm and foundation. (*Guidelines* Section 10.4.3.2.)

G. Gypsum Wallboard or Plaster Shear Walls

Plaster and gypsum wallboard can be removed and replaced with structural panel shear wall as required, and the new shear walls covered with gypsum wallboard. (*Guidelines* Section 10.4.3.3.)

10.3.4 Steel Braced Frames

10.3.4.1 System Concerns

If the strength of the braced frames is inadequate, more braced bays or shear wall panels can be added. The resulting lateral-force-resisting system must form a well-balanced system of braced frames that do not fail at their joints, and are properly connected to the floor diaphragms, and whose failure mode is yielding of braces rather than overturning. (FEMA 178 [BSSC, 1992a], Section 6.1.1.)

10.3.4.2 Stiffness of Diagonals

Diagonals with inadequate stiffness should be strengthened using the supplemental steel plates, or replaced with a larger and/or different type of section. Global stiffness can be increased by the addition of braced bays or shear wall panels. (FEMA 178 [BSSC, 1992a], Sections 6.1.2 and 6.1.3.)

10.3.4.3 Chevron or K-Bracing

Columns or horizontal girts can be added as needed to support the tension brace when the compression brace buckles, or the bracing can be revised to another system throughout the building. The beam elements can be strengthened with cover plates to provide them with the capacity to fully develop the unbalanced forces created by tension brace yielding. (FEMA 178 [BSSC, 1992a], Section 6.1.4.)

10.3.4.4 Braced Frame Connections

Column splices or other braced frame connections can be strengthened by adding plates and welds to ensure that they are strong enough to develop the connected members. Connection eccentricities that reduce member capacities can be eliminated, or the members can be strengthened to the required level by the addition of properly placed plates. Demands on the existing elements can be reduced by adding braced bays or shear wall panels. (FEMA 178 [BSSC, 1992a], Sections 6.1.5, 6.1.6, and 6.1.7.)

10.3.5 Diaphragms

10.3.5.1 Re-entrant Corners

New chords with sufficient strength to resist the required force can be added at the re-entrant corner. If a vertical lateral-force-resisting element exists at the re-entrant corner, a new shear collector element should be placed at the diaphragm, connected to the vertical element, to reduce tensile and compressive forces at the re-entrant corner. The same basic materials used in the diaphragm being strengthened should be used for the chord. (FEMA 178 [BSSC, 1992a], Section 7.1.1.)

10.3.5.2 Crossties

New crossties and wall connections can be added to resist the required out-of-plane wall forces and distribute these forces through the diaphragm. New strap plates and/or rod connections can be used to connect existing framing members together so they function as a crosstie in the diaphragm. (FEMA 178 [BSSC, 1992a], Section 7.1.2.)

10.3.5.3 Diaphragm Openings

New drag struts or diaphragm chords can be added around the perimeter of existing openings to distribute tension and compression forces along the diaphragm. The existing sheathing should be nailed to the new drag struts or diaphragm chords. In some cases it may also be necessary to: (1) increase the shear capacity of the diaphragm adjacent to the opening by overlaying the existing diaphragm with a wood structural panel, or (2) decrease the demand on the diaphragm by adding new vertical elements near the opening. (FEMA 178 [BSSC, 1992a], Sections 7.1.3 through 7.1.6.)

10.3.5.4 Diaphragm Stiffness/Strength

A. Board Sheathing

When the diaphragm does not have at least two nails through each board into each of the supporting members, and the lateral drift and/or shear demands on the diaphragm are not excessive, the shear capacity and stiffness of the diaphragm can be increased by adding nails at the sheathing boards. This method of upgrade is most often suitable in areas of low seismicity. In other cases, a new wood structural panel should be placed over the existing straight sheathing, and the joints of the wood structural panels placed so they are near the center of the sheathing boards or at a 45-degree angle to the joints between sheathing boards (see FEMA 172 [BSSC, 1992b], Section 3.5.1.2; ATC, [1981]; and FEMA 178 [BSSC, 1992a], Section 7.2.1).

B. Unblocked Diaphragms

The shear capacity of unblocked diaphragms can be improved by adding new wood blocking and nailing at the unsupported panel edges. Placing a new wood structural panel over the existing diaphragm will increase the shear capacity. Both of these methods will require the partial or total removal of existing flooring or roofing to place and nail the new overlay or nail the existing panels to the new blocking. Strengthening of the diaphragm is usually not necessary at the central area of the diaphragm where shear is low. In certain cases when the design loads are low, it may be possible to increase the shear capacity of unblocked diaphragms with sheet metal plates stapled on the underside of the existing wood panels. These plates and staples must be designed for all related shear and torsion caused by the details related to their installation. (FEMA 178 [BSSC, 1992a], Section 7.2.3.)

C. Spans

New vertical elements can be added to reduce the diaphragm span. The reduction of the diaphragm span will also reduce the lateral deflection and shear demand in the diaphragm. However, adding new vertical elements will result in a different distribution of shear demands. Additional blocking, nailing, or other rehabilitation measures may need to be provided at these areas. (FEMA 172, Section 3.4 and FEMA 178 [BSSC, 1992a], Section 7.2.2.)

D. Span-to-Depth Ratio

New vertical elements can be added to reduce the diaphragm span-to-depth ratio. The reduction of the diaphragm span-to-depth ratio will also reduce the lateral deflection and shear demand in the diaphragm. (Typical construction details and methods are discussed in FEMA 172, Section 3.4.) (FEMA 178 [BSSC, 1992a], Section 7.2.4.)

E. Diaphragm Continuity

The diaphragm discontinuity should in all cases be eliminated by adding new vertical elements at the diaphragm offset or the expansion joint (see FEMA 172, Section 3.4). In some cases, special details may be used to transfer shear across an expansion joint—while still allowing the expansion joint to function—thus eliminating a diaphragm discontinuity. (FEMA 178 [BSSC, 1992a], Section 7.2.5.)

F. Chord Continuity

If members such as edge joists, blocking, or wall top plates have the capacity to function as chords but lack connection, adding nailed or bolted continuity splices will provide a continuous diaphragm chord. New continuous steel or wood chord members can be added to the existing diaphragm where existing members lack sufficient capacity or no chord exists. New chord members can be placed at either the underside or topside of the diaphragm. In some cases, new vertical elements can be added to reduce the diaphragm span and stresses on any existing chord members (see FEMA 172, Section 3.5.1.3, and ATC-7). New chord connections should not be detailed such that they are the weakest element in the chord. (FEMA 178 [BSSC, 1992a], Section 7.2.6.)

10.3.6 Connections

10.3.6.1 Diaphragm/Wall Shear Transfer

Collector members, splice plates, and shear transfer devices can be added as required to deliver collector forces to the shear wall. Adding shear connectors from diaphragm to wall and/or to collectors will transfer shear. (See FEMA 172, Section 3.7 for Wood Diaphragms, 3.7.2 for concrete diaphragms, 3.7.3 for poured gypsum, and 3.7.4 for metal deck diaphragms.) (FEMA 178 [BSSC, 1992a], Sections 8.3.1 and 8.3.3.)

10.3.6.2 Diaphragm/Frame Shear Transfer

Adding collectors and connecting the framing will transfer loads to the collectors. Connections can be provided along the collector length and at the collector-to-frame connection to withstand the calculated forces (see FEMA 172, Sections 3.7.5 and 3.7.6). (FEMA 178 [BSSC, 1992a], Sections 8.3.2 and 8.3.3.)

10.3.6.3 Anchorage for Normal Forces

To account for inadequacies identified by FEMA 178 and in Section C10.3.6.3 of the *Commentary*, wall anchors can be added. Complications that may result from inadequate anchorage include cross-grain tension in wood ledgers, or failure of the diaphragm-to-wall connection, due to: (1) insufficient strength, number, or stability of anchors; (2) inadequate embedment of anchors; (3) inadequate development of anchors and straps into the diaphragm; and (4) deformation of anchors and their fasteners that permit diaphragm boundary connection pullout, or cross-grain tension in wood ledgers.

Existing anchors should be tested to determine load capacity and deformation potential including fastener slip, according to the requirements in Appendix C of FEMA 178 (BSSC, 1992a). Special attention should be given to the testing procedure to maintain a high level of quality control. Additional anchors should be provided as needed to supplement those that fail the test, as well as those needed to meet the FEMA 178 criteria. The quality of the rehabilitation depends greatly on the quality of the performed tests. (FEMA 178 [BSSC, 1992a], Sections 8.2.1 to 8.2.6; *Guidelines* Section 10.4.4.1.)

10.3.6.4 Girder-Wall Connections

The existing reinforcing must be exposed, and the connection modified as necessary. For out-of-plane loads, the number of column ties can be increased by jacketing the pilaster, or alternatively, by developing a second load path for the out-of-plane forces. Bearing length conditions can be addressed by adding bearing extensions. Frame action in welded connections can be mitigated by adding shear walls. (FEMA 178 [BSSC, 1992a], Sections 8.5.1 through 8.5.3.)

10.3.6.5 Precast Connections

The connections of chords, ties, and collectors can be upgraded to increase strength and/or ductility, providing alternative load paths for lateral forces. Upgrading can be achieved by such methods as adding confinement ties or increasing embedment. Shear walls can be added to reduce the demand on connections. (FEMA 178 [BSSC, 1992a], Section 4.4.2.)

10.3.6.6 Wall Panels and Cladding

It may be possible to improve the connection between the panels and the framing. If architectural or occupancy conditions warrant, the cladding can be replaced with a new system. The building can be stiffened with the addition of shear walls or braced frames, to reduce the drifts in the cladding elements. (FEMA 178 [BSSC, 1992a], Section 8.6.2.)

10.3.6.7 Light Gage Metal, Plastic, or Cementitious Roof Panels

It may be possible to improve the connection between the roof and the framing. If architectural or occupancy conditions warrant, the roof diaphragm can be replaced with a new one. Alternatively, a new diaphragm may be added, using rod braces or plywood above or below the existing roof, which remains in place. (FEMA 178 [BSSC, 1992a], Section 8.6.1.)

10.3.6.8 Mezzanine Connections

Diagonal braces, moment frames, or shear walls can be added at or near the perimeter of the mezzanine where bracing elements are missing, so that a complete and balanced lateral-force-resisting system is provided that meets the requirements of FEMA 178.

10.3.7 Foundations and Geologic Hazards

10.3.7.1 Anchorage to Foundations

For wood walls, expansion anchors or epoxy anchors can be installed by drilling through the wood sill to the concrete foundation at an appropriate spacing of four to six feet on center. Similarly, steel columns and wood posts can be anchored to concrete slabs or footings, using expansion anchors and clip angles. If the concrete or masonry walls and columns lack dowels, a concrete curb can be installed adjacent to the wall or column by drilling dowels and installing anchors into the wall that lap with dowels installed in the slab or footing. However, this curb can cause significant architectural problems. Alternatively, steel angles may be used with drilled anchors. The anchorage of shear wall boundary components can be challenging due to very high concentrated forces. (FEMA 178 [BSSC, 1992a], Sections 8.4.1 through 8.4.7.)

10.3.7.2 Condition of Foundations

All deteriorated and otherwise damaged foundations should be strengthened and repaired using the same materials and style of construction. Some conditions of material deterioration can be mitigated in the field, including patching of spalled concrete. Pest infestation or dry rot of wood piles can be very difficult to correct, and often require full replacement. The deterioration of these elements may have implications that extend beyond seismic safety and must be considered in the rehabilitation. (FEMA 178 [BSSC, 1992a], Sections 9.1.1 through 9.1.2.)

10.3.7.3 Overturning

Existing foundations can be strengthened as needed to resist overturning forces. Spread footings can be enlarged or additional piles, rock anchors, or piers added to deep foundations. It may also be possible to use grade beams or new wall elements to spread out overturning loads over a greater distance. Adding new lateral-load-resisting elements will reduce overturning effects of existing elements. (FEMA 178 [BSSC, 1992a], Section 9.2.1.)

10.3.7.4 Lateral Loads

As with overturning effects, the correction of lateral load deficiencies in the foundations of existing buildings is expensive and may not be justified by more realistic analysis procedures. For this reason, Systematic Rehabilitation is recommended for these cases. (FEMA 178 [BSSC, 1992a], Sections 9.2.1 through 9.2.5.)

10.3.7.5 Geologic Site Hazards

Site hazards other than ground shaking should be considered. Rehabilitation of structures subject to life safety hazards from ground failures is impractical, unless site hazards can be mitigated to the point where acceptable performance can be achieved. Not all ground failures need necessarily be considered as life safety hazards. For example, in many cases liquefaction beneath a building does not pose a life safety hazard; however, related lateral spreading can result in collapse of buildings with inadequate foundation strength. For this reason, the liquefaction potential and the related consequences should be thoroughly investigated for sites that do not satisfy the FEMA 178 statement. Further information on the evaluation of site hazards is provided in Chapter 4 of these *Guidelines*. (FEMA 178 [BSSC, 1992a], Sections 9.3.1 through 9.3.3.)

10.3.8 Evaluation of Materials and Conditions

10.3.8.1 General

Proper evaluation of the existing conditions and configuration of the existing building structure is an important aspect of Simplified Rehabilitation. As Simplified Rehabilitation is often concerned with specific deficiencies in a particular structural system, the evaluation can either be focused on affected structural elements and components, or be comprehensive and inclusive of the complete structure. If the degree of existing damage or deficiencies in a structure has not been established, the evaluation shall consist of a comprehensive inspection of gravity- and lateral-load-resisting systems that includes the following steps.

1. Verify existing data (e.g., accuracy of drawings).

2. Develop other needed data (e.g., measure and sketch building if necessary).
3. Verify the vertical and lateral systems.
4. Check the condition of the building.
5. Look for special conditions and anomalies.
6. Address the evaluation statements and goals during the inspection.
7. Perform material tests that are justified through a weighing of the cost of destructive testing and the cost of corrective work.

10.3.8.2 Condition of Wood

An inspection should be conducted to grade the existing wood and verify physical condition, using techniques from Section 10.3.8.1. Any damage or deterioration and its source must be identified. Wood that is significantly damaged due to splitting, decay, aging, or other phenomena must be removed and replaced. Localized problems can be eliminated by adding new appropriately sized reinforcing components extending beyond the damaged area and connecting to undamaged portions. Additional connectors between components should be provided to correct any discontinuous load paths. It is necessary to verify that any new reinforcing components or connectors will not be exposed to similar deterioration or damage. (FEMA 178 [BSSC, 1992a], Section 3.5.1.)

10.3.8.3 Overdriven Fasteners

Where visual inspection determines that extensive overdriving of fasteners exists in greater than 20% of the installed connectors, the fasteners and shear panels can generally be repaired through addition of a new same-sized fastener for every two overdriven fasteners. To avoid splitting because of closely spaced nails, it may be necessary to predrill to 90% of the nail shank diameter for installation of new nails. For other conditions, such as cases where the addition of new connectors is not possible or where component damage is suspected, further investigation shall be conducted using the guidance of Section 10.3.8.1. (FEMA 178 [BSSC, 1992a], Section 3.5.2.)

10.3.8.4 Condition of Steel

Should visual inspection or testing conducted per Section 10.3.8.1 reveal the presence of steel component or connection deterioration, further evaluation is needed. The source of the damage shall be identified and mitigative action shall be taken to preserve the remaining structure. In areas of significant deterioration, restoration of the material cross section can be performed by the addition of plates or other reinforcing. When sizing reinforcements, the design professional shall consider the effects of existing stresses in the original structure, load transfer, and strain compatibility. The demands on the deteriorated steel elements and components may also be reduced through careful addition of bracing or shear wall panels. (FEMA 178 [BSSC, 1992a], Section 3.5.3.)

10.3.8.5 Condition of Concrete

Should visual inspections or testing conducted per Section 10.3.8.1 reveal the presence of concrete component or reinforcing steel deterioration, further evaluation is needed. The source of the damage shall be identified and mitigative action shall be taken to preserve the remaining structure. Existing deteriorated material, including reinforcing steel, shall be removed to the limits defined by testing; reinforcing steel in good condition shall be cleaned and left in place for splicing purposes as appropriate. Cracks in otherwise sound material shall be evaluated to determine cause, and repaired as necessary using techniques appropriate to the source and activity level. (FEMA 178 [BSSC, 1992a], Sections 3.5.4 through 3.5.8.)

10.3.8.6 Post-Tensioning Anchors

Prestressed concrete systems may be adversely affected by cyclic deformations produced by earthquake motion. One rehabilitation process that may be considered is to add stiffness to the system. Another concern for these systems is the adverse effects of tendon corrosion. A thorough visual inspection of prestressed systems shall be performed to verify absence of concrete cracking or spalling, staining from embedded tendon corrosion, or other signs of damage along the tendon spans and at anchorage zones. If degradation is observed or suspected, more detailed evaluations will be required, as indicated in Chapter 6. Rehabilitation of these systems, except for local anchorage repair, should be in accordance with the Systematic Rehabilitation provisions in the balance of these *Guidelines*. Professionals with special prestressed concrete construction expertise should also be consulted for further interpretation of damage. (FEMA 178 [BSSC, 1992a], Section 3.5.5.)

10.3.8.7 Quality of Masonry

Should visual inspections or testing conducted per Section 10.3.8.1 reveal the presence of masonry component or construction deterioration, further evaluation is needed. Certain damage, such as degraded mortar joints or simple cracking, may be rehabilitated through repointing or rebuild. If the wall is repointed, care should be taken to ensure that the new mortar is compatible with the existing masonry units and mortar, and that suitable wetting is performed. The strength of the new mortar is critical to load-carrying capacity and seismic performance. Significant degradation should be treated as specified in Chapter 7 of these *Guidelines*. (FEMA 178 [BSSC, 1992a], Sections A4, 3.5.9, 3.5.10, and 3.5.11.)

10.4 Amendments to FEMA 178

Since the development and publication of FEMA 178 (BSSC, 1992a), significant earthquakes have occurred: the 1989 Loma Prieta earthquake in the San Francisco Bay area, the 1994 Northridge earthquake in the Los Angeles area, and the 1995 Hyogoken-Nanbu earthquake in the Kobe, Japan area. While each one generally validated the fundamental assumptions underlying the procedures, each also offered new insights into the potential weaknesses of certain lateral-force-resisting systems.

In the process of developing the *Guidelines* and *Commentary*, eight new potential deficiencies were identified and are developed below. They are presented in the same style as in FEMA 178 (BSSC, 1992a). Each is presented as a statement to be answered “True” or “False,” which permits rapid screening and identification of potential weak links. Each statement is followed by a paragraph of commentary written to identify the concern clearly. A suggested procedure for evaluating the potential weak link concludes each section, and should be carried out if the statement is found to be false. Completion of the procedure permits each potential deficiency to be properly evaluated and the actual deficiencies identified.

These eight new potential deficiencies should be considered as additions to the general list of building deficiencies (pages A3 to A16 of FEMA 178 [BSSC, 1992a]) and applied to the individual model buildings as indicated in Tables 10-3 through 10-20.

10.4.1 New Potential Deficiencies Related to Building Systems

10.4.1.1 Lateral Load Path at Pile Caps

Evaluation Statement: Pile caps are capable of transferring lateral and overturning forces between the structure and individual piles in the pile group.

Common problems with pile caps include a lack of top reinforcing in the pile cap. A loss of bond of pile and column reinforcing can occur when top cracks form during load reversals.

Procedure: Calculate the moment and shear capacity of the pile cap to transfer uplift and lateral forces based on the forces in FEMA 178 (BSSC, 1992a), from the point of application to each pile.

10.4.1.2 Deflection Compatibility

Evaluation Statement: Column and beam assemblies that are not part of the lateral-force-resisting system (i.e., gravity-load-resisting frames) are capable of accommodating imposed building drifts, including amplified drift caused by diaphragm deflections, without loss of their vertical-load-carrying capacity.

Frame components, especially columns, that are not specifically designed to participate in the lateral system will still undergo displacements associated with the overall seismic story drifts. If the columns are located far from the lateral-force-resisting elements, the added deflections due to semi-rigid floor diaphragms will increase the drifts. Stiff columns, designed for potentially high gravity loads, may develop significant bending moments due to the imposed drifts. The moment-axial force interaction may lead to brittle failures in nonductile columns, which could cause building collapse.

Procedure: Calculate expected drifts on the columns in frames that are not part of the lateral-force-resisting system, using procedures described in FEMA 178 (BSSC, 1992a), Section 2.4.4. Use cracked/transformed sections for all lateral-force-resisting concrete elements. Calculate additional drift from diaphragms by determining the deflection of the diaphragm at forces equal to those prescribed in FEMA 178, Chapter 2, for elements of structures. Evaluate the capacity of the non-lateral-force-resisting column and beam assemblies to undergo the combined drift, considering moment-axial interaction and column shear.

10.4.2 New Potential Deficiencies Related to Moment Frames

10.4.2.1 Moment-Resisting Connections

Evaluation Statement: All moment connections are able to develop the strength of the adjoining members or panel zones.

Connection failure is generally not ductile behavior. It is more desirable to have all inelastic action occur in the members rather than in the connections. The moment-resisting beam-column connection should provide for the development of the lesser of (1) the plastic girder strength in flexure or (2) the moment corresponding to the development of the panel zone shear strength, considering the effects of strain hardening and material overstrength. The deficiency is in the strength of the connections.

Procedure: At the time of this writing, this problem is the subject of the FEMA-funded effort carried out by the SAC Joint Venture, which is composed of the Structural Engineers Association of California (SEAOC), the Applied Technology Council (ATC), and California Universities for Research in Earthquake Engineering (CUREe), and which has produced interim guidance on the evaluation, repair, and rehabilitation of steel moment frames (SAC, 1995).

Using the latest guidelines, demonstrate by test or calculation that the connection meets the expected inelastic rotation demand on the joint, and that inelastic action is not concentrated in the vicinity of welds at the column face.

10.4.2.2 Short Captive Columns

Evaluation Statement: There are no columns with height-to-depth ratios less than 75% of the nominal height-to-depth ratios of the typical columns at that level.

Short captive columns (which are usually not designed as part of the primary lateral-load-resisting system) tend to attract shear forces because of their high stiffness relative to other lateral-force-resisting vertical elements at that story level. Significant damage has been observed in parking structure columns adjacent to ramping slabs, even in structures with shear walls. Captive column behavior may also be found in buildings with clerestory windows, in buildings where columns are partially braced by masonry or concrete nonstructural construction, and in buildings with improperly designed mezzanines.

Procedure: Calculate the anticipated story drift, and determine the shear (V_e) demand in the short column caused by the drift ($V_e = 2M/L$). Compare V_e with the member nominal shear capacity (V_n) calculated in accordance with ACI (1989) Chapter 21. The ratio V_e / V_n should be less than or equal to 1.0.

10.4.3 New Potential Deficiencies Related to Shear Walls

10.4.3.1 Narrow Wood Shear Walls

Evaluation Statement: Narrow wood shear walls with an aspect ratio greater than two to one do not resist forces developed in the building.

Most of the deformation of the narrow shear walls occurs at the base, and consists of sliding of the sill plate and stretching of hold-down attachments. Splitting of the end studs at the attachment of hold-downs is also a common failure. Narrow shear walls are relatively flexible and thus tend to take less shear than would be anticipated when compared to wider shear walls. This results in greater loading of the shear walls with lower height-to-width ratios and less load in the narrow walls.

Procedure: Determine the shear capacity of the wall and related overturning demand. Verify that shear and overturning can be transferred to the foundation within allowable stresses calculated in accordance with FEMA 178 (BSSC, 1992a).

10.4.3.2 Stucco (Exterior Plaster) Shear Walls

Evaluation Statement: Multistory buildings do not rely on exterior stucco walls as the primary lateral-force-resisting system.

Exterior stucco plaster walls are often used (intentionally and unintentionally) for resisting lateral earthquake loads. Stucco is relatively stiff and brittle, with low shear resistance value. Differential foundation movement and earthquake shaking cause cracking of the stucco and loss of lateral strength. The cracking can range from minor to severe. Sometimes the stucco delaminates from the framing and the lateral-force-resisting system is lost. Multistory buildings shall not rely on stucco walls as the primary lateral-force-resisting system since there is not enough available strength.

Procedure: Inspect stucco clad buildings to determine if there is a lateral system such as plywood or diagonal sheathing in at least all but the top floor. Where exterior plaster is utilized and there is a supplemental system, verify that the wire reinforcing is attached directly to the wall framing and the wire is completely embedded in the plaster material. Verify that lateral loads do not exceed 100 pounds per linear foot.

10.4.3.3 Gypsum Wallboard or Plaster Shear Walls

Evaluation Statement: Interior plaster or gypsum wallboard is not being used for shear walls on buildings over one story in height.

Gypsum wallboard or gypsum plaster sheathing tends to be easily damaged by differential foundation movement or earthquake shaking. Most residential buildings have numerous walls constructed with plaster or gypsum wallboard. Though the capacity of these walls is low, the amount of wall is often high. As a result, plaster and gypsum wallboard walls may provide adequate resistance to moderate earthquake shaking. The problem that can occur is incompatibility with other lateral-forcing-resisting elements. For example, narrow plywood shear walls are more flexible than long stiff plaster walls; as a result, the plaster or gypsum walls will take all the load until they fail and then the plywood walls will start to resist the lateral loads. Plaster or gypsum wallboard walls should not be used for shear walls except for one-story buildings or on the top story of multistory buildings.

Procedure: Determine the walls with plaster or gypsum sheathing that would be required to resist lateral earthquake forces (i.e., earthquake loads would have to pass through these walls), due to the location of the walls in the building. Verify that all walls have been properly constructed with nailing required by FEMA 222A (BSSC, 1995), and that loads are within allowable limits. Remove gypsum wallboard and plaster as required, and replace with panel shear walls. Cover the new shear walls with gypsum wallboard and plaster.

10.4.4 New Potential Deficiencies Related to Connections

10.4.4.1 Stiffness of Wall Anchors

Evaluation Statement: Anchors of heavy concrete or masonry walls to wood structural elements are installed taut and are stiff enough to prevent movement between the wall and roof. If bolts are used, the bolt holes in both the connector and framing are a maximum of 1/16" larger than the bolt diameter.

The small separation that can occur between the wall and roof sheathing, due to anchors that are not taut, requires movement before taking hold and can result in an out-of-plane failure of the ledger support. Bolts in oversized holes can also cause slippage and separation between the wall and framing.

Procedure: Field check that no anchor has a twist, kink, or offset, and that no anchor is otherwise installed such that some separation must occur prior to its taking hold, and that such movement will lead to a perpendicular-to-grain bending failure in the wood ledger. Remove a representative sample of bolts and verify that the holes are not oversized. For oversized holes, replace bolts and fill gaps with epoxy or other suitable filler.

10.5 FEMA 178 Deficiency Statements

No guidelines are provided for this section. See the *Commentary* for a complete list—augmented with the eight new deficiency statements from Section 10.4, above—presented in a logical, combined order.

10.6 Definitions

Boundary component (boundary member): A member at the perimeter (edge or opening) of a shear wall or horizontal diaphragm that provides tensile and/ or compressive strength.

Column (or beam) jacketing: A method in which a concrete column or beam is covered with a steel or concrete "jacket" in order to strengthen and/or repair the member by confining the concrete.

Coupling beam: Flexural member that ties or couples adjacent shear walls acting in the same plane. A coupling beam is designed to yield and dissipate inelastic energy, and, when properly detailed and proportioned, has a significant effect on the overall stiffness of the coupled wall.

Crosstie: A beam or girder that spans across the width of the diaphragm, accumulates the wall loads, and transfers them, over the full depth of the diaphragms, into the next bay and onto the nearest shear wall or frame.

Diaphragm chord: A diaphragm component provided to resist tension or compression at the edges of the diaphragm.

Drag strut: A component parallel to the applied load that collects and transfers diaphragm shear forces to the vertical lateral-force-resisting components or elements, or distributes forces within a diaphragm.

Flexible diaphragm: A diaphragm consisting of one of the following systems: plywood sheathing, spaced timber sheathing, straight timber sheathing, diagonal timber sheathing, metal deck without concrete fill, corrugated transit panels, or steel rod bracing or other steel bracing using light members such as angles or split tees.

Inter-story drift: The relative horizontal displacement of two adjacent floors in a building. Inter-story drift can also be expressed as a percentage of the story height separating the two adjacent floors.

Load path: A path that seismic forces pass through to the foundation of the structure and, ultimately, to the soil. Typically, the load travels from the diaphragm through connections to the vertical lateral-force-resisting elements, and then proceeds to the foundation by way of additional connections.

Model Building Type: Fifteen common building types used to categorize expected deficiencies, reasonable rehabilitation methods, and estimated costs. See Table 10-2 for descriptions of Model Building Types.

Narrow wood shear wall: Wood shear walls with an aspect ratio (height to width) greater than two to one. These walls are relatively flexible and thus tend to be incompatible with other building components, thereby taking less shear than would be anticipated when compared to wider walls.

Noncompact member: A steel section in compression whose width-to-thickness ratio does not meet the limiting values for compactness, as shown in Table B5.1 of AISC (1986).

Overturning: Action resulting when the moment produced at the base of vertical lateral-force-resisting elements is larger than the resistance provided by the foundation's uplift resistance and building weight.

Panel zone: Area of a column at the beam-to-column connection delineated by beam and column flanges.

Plan irregularity: Horizontal irregularity in the layout of vertical lateral-force-resisting elements, producing a misalignment between the center of mass and center of rigidity that typically results in significant torsional demands on the structure.

Pounding: Two adjacent buildings coming into contact during earthquake excitation because they are too close together and/or exhibit different dynamic deflection characteristics.

Re-entrant corner: Plan irregularity in a diaphragm, such as an extending wing, plan inset, or E-, T-, X-, or L-shaped configuration, where large tensile and compressive forces can develop.

Redundancy: Quality of having alternative paths in the structure by which the lateral forces are resisted, allowing the structure to remain stable following the failure of any single element.

Rehabilitation Objective: A statement of the desired limits of damage or loss for a given seismic demand, usually selected by the owner, engineer, and/or relevant public agencies. (See Chapter 2.)

Repointing: A method of repairing a cracked or deteriorating mortar joint in masonry. The damaged or deteriorated mortar is removed and the joint is refilled with new mortar.

Short captive column: Columns with height-to-depth ratios less than 75% of the nominal height-to-depth ratios of the typical columns at that level. These columns, which may not be designed as part of the primary lateral-load-resisting system, tend to attract shear forces because of their high stiffness relative to adjacent elements.

Simplified Rehabilitation Method: An approach, applicable to some types of buildings and Rehabilitation Objectives, in which analyses of the entire building's response to earthquake hazards are not required.

Stiff diaphragm: A diaphragm consisting of one of the following systems: monolithic reinforced concrete slabs, precast concrete slabs or planks bonded together by a reinforced topping slab or by welded inserts, concrete-filled metal deck, or masonry arches with or without concrete fill or topping.

Strong column-weak beam: A connection required to localize damage and control drift; the capacity of the column in any moment frame joint must be greater than that of the beams, to ensure inelastic action in the beams.

Strong back system: A secondary system, such as a frame, commonly used to provide out-of-plane support for an unreinforced or under-reinforced masonry wall.

Systematic Rehabilitation Method: An approach to rehabilitation in which complete analysis of the building's response to earthquake shaking is performed.

Vertical irregularity: A discontinuity of strength, stiffness, geometry, or mass in one story with respect to adjacent stories.

10.7 Symbols

L Length of the column

M Moment expected in the column at maximum expected drift

V_e Shear demand in the column caused by the drift

V_n Nominal shear capacity of a column

10.8 References

ACI, 1989, *Building Code Requirements for Reinforced Concrete*, ACI 318-89, American Concrete Institute, Detroit, Michigan.

ATC, 1981, *Guidelines for the Design of Horizontal Wood Diaphragms*, Report No. ATC-7, Applied Technology Council, Redwood City, California.

BSSC, 1992a, *NEHRP Handbook for the Seismic Evaluation of Existing Building*s, developed by the Building Seismic Safety Council for the Federal Emergency Management Agency (Report No. FEMA 178), Washington, D.C.

BSSC, 1992b, *NEHRP Handbook of Techniques for the Seismic Rehabilitation of Existing Buildings*, developed by the Building Seismic Safety Council for the Federal Emergency Management Agency (Report No. FEMA 172), Washington, D.C.

BSSC, 1995, *NEHRP Recommended Provisions for Seismic Regulations for New Buildings, 1994 Edition, Part 1: Provisions and Part 2: Commentary*, prepared by the Building Seismic Safety Council for the Federal Emergency Management Agency (Report Nos. FEMA 222A and 223A), Washington, D.C.

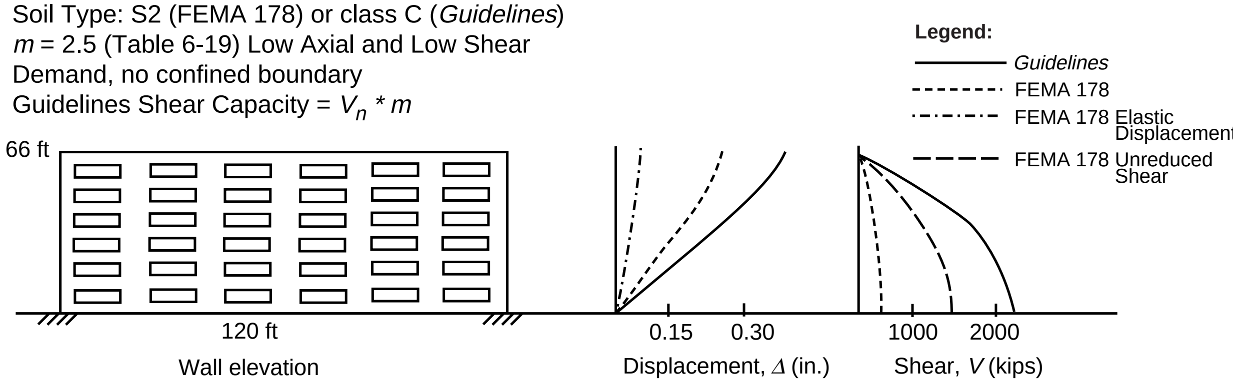
Masonry Standards Joint Committee (MSJC), 1995, *Building Code Requirements for Masonry Structures*, ACI 530-95/ASCE 5-95/TMS 402-95, American Concrete Institute, Detroit, Michigan; American Society of Civil Engineers, New York, New York; and the Masonry Society, Boulder, Colorado.

SAC, 1995, *Interim Guidelines: Evaluation, Repair, Modification and Design of Steel Moment Frames*, Report No. SAC-95-02, developed by the SAC Joint Venture (the Structural Engineers Association of California [SEAOC], the Applied Technology Council [ATC], and California Universities for Research in Earthquake Engineering [CUREe]) for the Federal Emergency Management Agency (Report No. FEMA 267), Washington, D.C.

Six-Story Reinforced Concrete Shear Wall Building in Low-Seismicity Region

- Low-seismicity region (NEHRP Region 1, BSSC [1988])
- 120-foot square in plan with 8-inch-thick reinforced concrete exterior walls and 9-inch-thick reinforced concrete floor slabs
- Life Safety Performance Level
- 10%/50 year ground motion (*Guidelines*)
  Soil Type: S2 (FEMA 178) or class C (*Guidelines*)
- *m* = 2.5 (Table 6-19) Low Axial and Low Shear Demand, no confined boundary
- Guidelines Shear Capacity = $V_n * m$

	Base shear	Maximum pier shear		
		Demand	Capacity	Dema
FEMA 178	300 kips	33 psi	276 psi	0.12
Guidelines	2117 kips	230 psi	690 psi	0.21



Three-Story Reinforced Concrete Shear Wall Building in High-Seismicity Region

- High-seismicity region (NEHRP Region 7, BSSC [1988])
- 120-foot square in plan with 8-inch-thick reinforced concrete exterior walls and 9-inch-thick reinforced concrete floor slabs
- Life Safety Performance Level
- 10%/50 year ground motion (*Guidelines*)
- Soil Type: S2 (FEMA 178) or class C (*Guidelines*)

	Base shear	Maximum pier shear		
		Demand	Capacity	Dema
FEMA 178	1157 kips	126 psi	276 psi	0.46
Guidelines	6091 kips	659 psi	829 psi	0.79

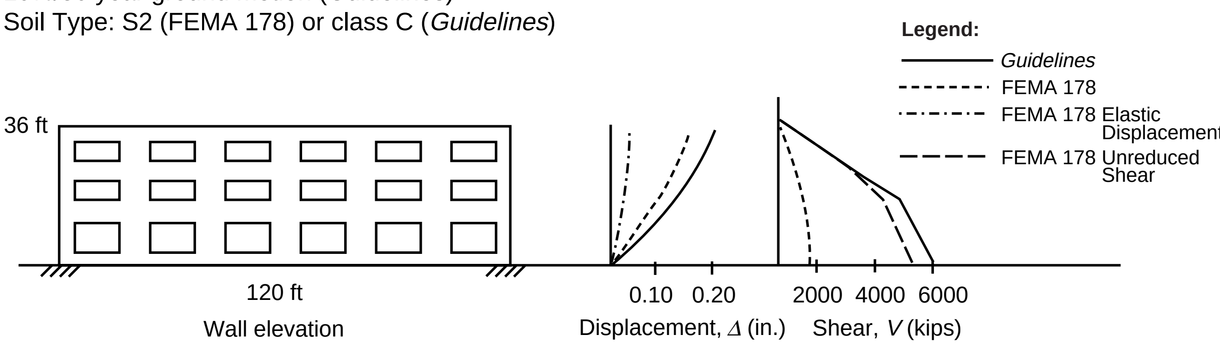


Figure 10-1 Comparison of FEMA 178 (BSSC, 1992a) and Guidelines Acceptance Criteria

Table 10-1 Limitations on Use of Simplified Rehabilitation Method

Model Building Type[2]	Maximum Building Height in Stories by Seismic Zone[1] for Use of Simplified Rehabilitation Method		
	Low	**Moderate**	**High**
Wood Frame			
Light (W1)	3	3	2
Multistory Multi-Unit Residential (W1A)	3	3	2
Commercial and Industrial (W2)	3	3	2
Steel Moment Frame			
Stiff Diaphragm (S1)	6	4	3
Flexible Diaphragm (S1A)	4	4	3
Steel Braced Frame			
Stiff Diaphragm (S2)	6	4	3
Flexible Diaphragm (S2A)	3	3	3
Steel Light Frame (S3)	2	2	2
Steel Frame with Concrete Shear Walls (S4)	6	4	3
Steel Frame with Infill Masonry Shear Walls			
Stiff Diaphragm (S5)	3	3	(shaded)
Flexible Diaphragm (S5A)	3	3	(shaded)
Concrete Moment Frame (C1)	3	(shaded)	(shaded)
Concrete Shear Walls			
Stiff Diaphragm (C2)	6	4	3
Flexible Diaphragm (C2A)	3	3	3
Concrete Frame with Infill Masonry Shear Walls			
Stiff Diaphragm (C3)	3	(shaded)	(shaded)
Flexible Diaphragm (C3A)	3	(shaded)	(shaded)
Precast/Tilt-up Concrete Shear Walls			
Flexible Diaphragm (PC1)	3	2	2
Stiff Diaphragm (PC1A)	3	2	2
Precast Concrete Frame			
With Shear Walls (PC2)	3	2	(shaded)
Without Shear Walls (PC2A)	(shaded)	(shaded)	(shaded)
Reinforced Masonry Bearing Walls			
Flexible Diaphragm (RM1)	3	3	3
Stiff Diaphragm (RM2)	6	4	3

(shaded) = Use of Simplified Rehabilitation Method not appropriate.

1. Seismic Zones are defined in Chapter 2 of the *Guidelines*.
2. Buildings with different types of flexible diaphragms may be considered to have flexible diaphragms.
Multistory buildings having stiff diaphragms at all levels except the roof may be considered as having stiff diaphragms.
Buildings having both flexible and stiff diaphragms, or having diaphragm systems that are neither flexible nor stiff, in accordance with this chapter, shall be rehabilitated using the Systematic Method.

Table 10-1 Limitations on Use of Simplified Rehabilitation Method (continued)

Model Building Type[2]	Maximum Building Height in Stories by Seismic Zone[1] for Use of Simplified Rehabilitation Method		
	Low	**Moderate**	**High**
Unreinforced Masonry Bearing Walls			
Flexible Diaphragm (URM)	3	3	2
Stiff Diaphragm (URMA)	3	3	2

= Use of Simplified Rehabilitation Method not appropriate.

1. Seismic Zones are defined in Chapter 2 of the *Guidelines*.
2. Buildings with different types of flexible diaphragms may be considered to have flexible diaphragms.
Multistory buildings having stiff diaphragms at all levels except the roof may be considered as having stiff diaphragms.
Buildings having both flexible and stiff diaphragms, or having diaphragm systems that are neither flexible nor stiff, in accordance with this chapter, shall be rehabilitated using the Systematic Method.

Table 10-2 Description of Model Building Types

Model Building Type designations are similar to those used in FEMA 178 (BSSC, 1992a). Designations ending in the letter A indicate types added to the FEMA 178 list. Refer to FEMA 178 for additional information.

Building Type 1—Wood, Light Frame	
Type W1:	These building are typically single- or multiple-family dwellings of one or more stories. The essential structural character of this type is repetitive framing by wood joists on wood studs. Loads are light and spans are small. These buildings may have relatively heavy chimneys and may be partially or fully covered with veneer. Most of these buildings are not engineered; however, they usually have the components of a lateral-force-resisting system even though it may be incomplete. Lateral loads are transferred by diaphragms to shear walls. The diaphragms are roof panels and floors. Shear walls are exterior walls sheathed with plank siding, stucco, plywood, gypsum board, particleboard, or fiberboard. Interior partitions are sheathed with plaster or gypsum board.
Type W1A:	Similar to W1 buildings, but are typically multistory multi-unit residential structures, often with open front garages at the first story.
Building Type 2—Wood, Commercial and Industrial	
Type W2:	These buildings usually are commercial or industrial buildings with a floor area of 5,000 square feet or more and few, if any, interior walls. The essential structural character is framing by beams on columns. The beams may be glulam beams, steel beams, or trusses. Lateral forces usually are resisted by wood diaphragms and exterior walls sheathed with plywood, stucco, plaster, or other paneling. The walls may have rod bracing. Large openings for stores and garages often require post-and-beam framing. Lateral force resistance on those lines can be achieved with steel rigid frames or diagonal bracing.
Building Type 3—Steel Moment Frame	
Type S1:	These buildings have a frame of steel columns and beams. In some cases, the beam-column connections have very small moment-resisting capacity, but in other cases some of the beams and columns are fully developed as moment frames to resist lateral forces. Usually the structure is concealed on the outside by exterior walls, which can be of almost any material (curtain walls, brick masonry, or precast concrete panels), and on the inside by ceilings and column furring. Lateral loads are transferred by diaphragms to moment-resisting frames. The diaphragms are typically concrete or metal deck with concrete fill, and are considered stiff with respect to the frames. The frames develop their stiffness by full or partial moment connections. The frames can be located almost anywhere in the building. Usually the columns have their strong directions oriented so that some columns act primarily in one direction while the others act in the other direction, and the frames consist of lines of strong columns and their intervening beams. Steel moment frame buildings are typically more flexible than shear wall buildings. This low stiffness can result in large inter-story drifts that may lead to extensive nonstructural damage.
Type S1A:	Similar to Type S1, except that diaphragms are typically wood, tile arch, or bare metal deck and are considered flexible with respect to the frames. Concrete or metal deck with concrete fill diaphragms may be considered flexible if the span-to-depth ratio between lines of moment frames is high. Steel frame with wood or tile floors is more common in older styles of construction.
Building Type 4—Steel Braced Frame	
Type S2:	These buildings are similar to Type 3 (S1) buildings, except that the vertical components of the lateral-force-resisting system are braced frames rather than moment frames.
Type S2A:	These buildings are similar to Type 3 (S1A) buildings, except that the vertical components of the lateral-force-resisting system are braced frames rather than moment frames.
Building Type 5—Steel Light Frame	
Type S3:	These buildings are pre-engineered and prefabricated with transverse rigid frames. The roof and walls consist of lightweight panels. The frames are designed for maximum efficiency, often with tapered beam and column sections built up of light plates. The frames are built in segments and assembled in the field with bolted joints. Lateral loads in the transverse direction are resisted by the rigid frames with loads distributed to them by shear elements. Loads in the longitudinal direction are resisted entirely by shear elements. The shear elements can be either the roof and wall sheathing panels, an independent system of tension-only rod bracing, or a combination of panels and bracing.

Table 10-2 Description of Model Building Types (continued)

Model Building Type designations are similar to those used in FEMA 178 (BSSC, 1992a). Designations ending in the letter A indicate types added to the FEMA 178 list. Refer to FEMA 178 for additional information.

Building Type 6—Steel Frame with Concrete Shear Walls	
Type S4:	The shear walls in these buildings are cast-in-place concrete and may be bearing walls. The steel frame is designed for vertical loads only. Lateral loads are transferred by diaphragms—typically of cast-in-place concrete—to the shear walls. The steel frame may provide a secondary lateral-force-resisting system depending on the stiffness of the frame and the moment capacity of the beam-column connections. In modern “dual” systems, the steel moment frames are designed to work together with the concrete shear walls in proportion to their relative rigidities. In this case, the walls would be evaluated under this building type and the frames would be evaluated under Building Type 3, Steel Moment Frame.
Building Type 7—Steel Frame with Infill Masonry Shear Walls	
Type S5:	This is one of the older types of building. The infill walls usually are offset from the exterior frame members, wrap around them, and present a smooth masonry exterior with no indication of the frame. Solidly infilled masonry panels act as a diagonal compression strut between the intersections of the moment frame. If the walls do not fully engage the frame members (i.e., lie in the same plane), the diagonal compression struts will not develop. The peak strength of the diagonal strut is determined by the diagonal tensile stress capacity of the masonry panel. The post-cracking strength is determined by an analysis of a moment frame that is partially restrained by the cracked infill. The analysis should be based on published research and should treat the system as a composite of a frame and the infill. An analysis that attempts to treat the system as a frame and shear wall is not capable of assuring compatibility. Diaphragms are typically concrete or tile arch with short spans between infill walls, and are considered stiff with respect to the walls.
Type S5A:	Similar to Type S5, except that diaphragms either are wood, or contain concrete or tile floors with a high span-to-depth ratio between infill walls, and are considered flexible with respect to the walls.
Building Type 8—Concrete Moment Frame	
Type C1:	These buildings are similar to Type 3 (S1) buildings, except that the frames are of concrete. Some older concrete frames may be proportioned and detailed such that brittle failure can occur. There is a large variety of frame systems. Buildings in zones of low seismicity or older buildings in zones of high seismicity can have frame beams that have broad shallow cross sections or are simply the column strips of flat slabs. Modern frames in zones of high seismicity are detailed for ductile behavior and the beams and columns have definitely regulated proportions. Diaphragms are typically concrete.
Building Type 9—Concrete Shear Walls	
Type C2:	The vertical components of the lateral-force-resisting system in these buildings are concrete shear walls that are usually bearing walls. In older buildings, the walls often are quite extensive and the wall stresses are low, but reinforcing is light. When remodeling calls for enlarging the windows, the strength of the modified walls becomes a critical concern. In newer buildings, the shear walls often are limited in extent, thus generating concerns about boundary members and overturning forces. Diaphragms are typically cast-in-place concrete slabs with or without beams and are considered stiff with respect to the walls.
Type C2A:	Similar to Type C2, except that diaphragms are typically wood and are considered flexible with respect to the frames. This is typically evident in older styles of construction. Concrete diaphragms may be considered flexible if the span-to-depth ratio between shear walls is high. This is common in parking structures and in buildings with narrow aspect ratios.
Building Type 10—Concrete Frame with Infill Masonry Shear Walls	
Type C3:	These buildings are similar to Type 7 (S5) buildings except that the frame is of reinforced concrete. The analysis of this building is similar to that recommended for Type 7 (S5), except that the shear strength of the concrete columns, after cracking of the infill, may limit the semiductile behavior of the system. Research that is specific to confinement of the infill by reinforced concrete frames should be used for the analysis.
Type C3A:	Similar to Type C3, except that diaphragms either are wood, or contain concrete or tile floors with a high span-to-depth ratios between infill walls, and are considered flexible with respect to the walls.

Table 10-2 Description of Model Building Types (continued)

Model Building Type designations are similar to those used in FEMA 178 (BSSC, 1992a). Designations ending in the letter A indicate types added to the FEMA 178 list. Refer to FEMA 178 for additional information.

Building Type 11—Precast/Tilt-up Concrete Shear Walls	
Type PC1:	These buildings have a wood or metal deck roof diaphragm, which often is very large, that distributes lateral forces to precast concrete shear walls and is considered flexible with respect to the walls. They may also have precast concrete diaphragms if the span-to-depth ratio between walls is very high or there is no topping slab. The walls are thin but relatively heavy, while the roofs are relatively light. Older buildings often have inadequate connections for anchorage of the walls to the roof for out-of-plane forces, and the panel connections often are brittle. Tilt-up buildings often have more than one story. Walls may have numerous openings for doors and windows of such size that the wall looks more like a frame than a shear wall.
Type PC1A:	Similar to Type PC1, except that diaphragms are precast or cast-in-place concrete with small span-to-depth ratios, and are considered stiff with respect to the walls.
Building Type 12—Precast Concrete Frame	
Type PC2:	These buildings contain floor and roof diaphragms typically composed of precast concrete elements with or without cast-in-place concrete topping slabs. The diaphragms are supported by precast concrete girders and columns. The girders often bear on column corbels. Closure strips between precast floor elements and beam-column joints usually are cast-in-place concrete. Welded steel inserts often are used to interconnect precast elements. Lateral loads are resisted by precast or cast-in-place concrete shear walls. Buildings with precast frames and concrete shear walls should perform well if the details used to connect the structural elements have sufficient strength and displacement capacity; however, in some cases the connection details between the precast elements have negligible ductility.
Type PC2A:	Similar to Type PC2, except that lateral loads are resisted by the concrete frames directly without the presence of shear walls. This type of construction is not permitted in regions of high seismicity.
Building Type 13—Reinforced Masonry Bearing Walls with Flexible Diaphragms	
Type RM1:	These buildings have bearing and shear walls of reinforced brick or concrete-block masonry, which are the vertical elements in the lateral-force-resisting system. The floors and roofs are framed either with wood joists and beams with plywood or straight or diagonal sheathing, or with steel beams with metal deck with or without a concrete fill. Wood floor framing is supported by interior wood posts or steel columns; steel beams are supported by steel columns.
Building Type 14—Reinforced Masonry Bearing Walls with Stiff Diaphragms	
Type RM2:	These buildings have walls similar to those of Type 13 (RM1) buildings, but the roof and floors are composed of precast concrete elements such as planks or T-beams, and the precast roof and floor elements are supported on interior beams and columns of steel or concrete (cast-in-place or precast). The precast horizontal elements often have a cast-in-place topping.
Building Type 15—Unreinforced Masonry Bearing Walls	
Type URM:	These buildings include structural elements that vary depending on the building's age and, to a lesser extent, its geographic location. In buildings built before 1900, the majority of floor and roof construction consists of wood sheathing supported by wood subframing. In buildings built after 1950, unreinforced masonry building with wood floors usually have plywood rather than board sheathing. The diaphragms are considered flexible with respect to the walls. The perimeter walls, and possibly some interior walls, are unreinforced masonry. The walls may or may not be anchored to the diaphragms. Ties between the walls and diaphragms are more common for the bearing walls than for walls that are parallel to the floor framing. Roof ties usually are less common and more erratically spaced than those at the floor levels. Interior partitions that interconnect the floors and roof can have the effect of reducing diaphragm displacements.
Type URMA:	Similar to Type URM, except that the floors are cast-in-place concrete supported by the unreinforced masonry walls and/or steel or concrete interior framing. This is more common in older, large, multistory buildings. In regions of lower seismicity, buildings of this type constructed more recently can include floor and roof framing that consists of metal deck and concrete fill supported by steel framing elements. The diaphragms are considered stiff with respect to the walls.

Table 10-3 W1: Wood Light Frame

Typical Deficiencies

Load Path
Redundancy
Vertical Irregularities

Shear Walls in Wood Frame Buildings
- Shear Stress
- Openings
- Wall Detailing
- Cripple Walls
- Narrow Wood Shear Walls
- Stucco Shear Walls
- Gypsum Wallboard or Plaster Shear Walls

Diaphragm Openings
Diaphragm Stiffness/Strength
- Spans
- Diaphragm Continuity

Anchorage to Foundations
Condition of Foundations
Geologic Site Hazards

Condition of Wood

Table 10-4 W1A: Multistory, Multi-Unit, Wood Frame Construction

Typical Deficiencies

Load Path
Redundancy
Vertical Irregularities

Shear Walls in Wood Frame Buildings
- Shear Stress
- Openings
- Wall Detailing
- Cripple Walls
- Narrow Wood Shear Walls
- Stucco Shear Walls
- Gypsum Wallboard or Plaster Shear Walls

Diaphragm Openings
Diaphragm Stiffness/Strength
- Spans
- Diaphragm Continuity

Anchorage to Foundations
Condition of Foundations
Geologic Site Hazards

Condition of Wood

Table 10-5 W2: Wood, Commercial, and Industrial

Typical Deficiencies

Load Path
Redundancy
Vertical Irregularities

Shear Walls in Wood Frame Buildings
- Shear Stress
- Openings
- Wall Detailing
- Cripple Walls
- Narrow Wood Shear Walls
- Stucco Shear Walls
- Gypsum Wallboard or Plaster Shear Walls

Diaphragm Openings
Diaphragm Stiffness/Strength
- Sheathing
- Unblocked Diaphragms
- Spans
- Span-to-Depth Ratio
- Diaphragm Continuity
- Chord Continuity

Anchorage to Foundations
Condition of Foundations
Geologic Site Hazards

Condition of Wood

Table 10-6 S1 and S1A: Steel Moment Frames with Stiff or Flexible Diaphragms

Typical Deficiencies

Load Path
Redundancy
Vertical Irregularities
Plan Irregularities
Adjacent Buildings
Lateral Load Path at Pile Caps

Steel Moment Frames
- Drift Check
- Frame Concerns
- Strong Column-Weak Beam
- Connections

Re-entrant Corners
Diaphragm Openings
Diaphragm Stiffness/Strength

Diaphragm/Frame Shear Transfer

Anchorage to Foundations
Condition of Foundations
Overturning
Lateral Loads
Geologic Site Hazards

Condition of Steel

Table 10-7 S2 and S2A: Steel Braced Frames with Stiff or Flexible Diaphragms

Typical Deficiencies
Load Path Redundancy Vertical Irregularities Plan Irregularities Lateral Load Path at Pile Caps
Stress Level Stiffness of Diagonals Chevron or K-Bracing Braced Frame Connections
Re-entrant Corners Diaphragm Openings Diaphragm Stiffness/Strength
Diaphragm/Frame Shear Transfer
Anchorage to Foundations Condition of Foundations Overturning Lateral Loads Geologic Site Hazards
Condition of Steel

Table 10-8 S3: Steel Light Frames

Typical Deficiencies
Load Path Redundancy Vertical Irregularities Plan Irregularities
Steel Moment Frames   Frame Concerns
Masonry Shear Walls   Infill Walls

Steel Braced Frames
   Stress Level
   Braced Frame Connections

Re-entrant Corners Diaphragm Openings
Diaphragm/Frame Shear Transfer Wall Panels and Cladding Light Gage Metal, Plastic, or Cementitious Roof Panels
Anchorage to Foundations Condition of Foundations Geologic Site Hazards
Condition of Steel

Table 10-9 S4: Steel Frames with Concrete Shear Walls

Typical Deficiencies
Load Path Redundancy Vertical Irregularities Plan Irregularities Lateral Load Path at Pile Caps

Cast-in-Place Concrete Shear Walls
   Shear Stress
   Overturning
   Coupling Beams
   Boundary Component Detailing
   Wall Reinforcement

Re-entrant Corners Diaphragm Openings Diaphragm Stiffness/Strength
Diaphragm/Wall Shear Transfer
Anchorage to Foundations Condition of Foundations Overturning Lateral Loads Geologic Site Hazards
Condition of Steel Condition of Concrete

Table 10-10 S5, S5A: Steel Frames with Infill Masonry Shear Walls and Stiff or Flexible Diaphragms

Typical Deficiencies
Load Path Redundancy Vertical Irregularities Plan Irregularities Lateral Load Path at Pile Caps
Frames Not Part of the Lateral Force Resisting System   Complete Frames

Masonry Shear Walls
   Reinforcing in Masonry Walls
   Shear Stress
   Reinforcing at Openings
   Unreinforced Masonry Shear Walls
   Proportions, Solid Walls
   Infill Walls

Re-entrant Corners Diaphragm Openings Diaphragm Stiffness/Strength   Span/Depth Ratio
Diaphragm/Wall Shear Transfer Anchorage for Normal Forces
Anchorage to Foundations Condition of Foundations Overturning Lateral Loads Geologic Site Hazards
Condition of Steel Quality of Masonry

Table 10-11 C1: Concrete Moment Frames

Typical Deficiencies
Load Path Redundancy Vertical Irregularities Plan Irregularities Adjacent Buildings Lateral Load Path at Pile Caps Deflection Compatibility

Concrete Moment Frames
   Quick Checks, Frame and Nonductile Detail Concerns
   Precast Moment Frame Concerns
Frames Not Part of the Lateral Force Resisting System
   Short “Captive” Columns

Re-entrant Corners Diaphragm Openings Diaphragm Stiffness/Strength
Diaphragm/Frame Shear Transfer Precast Connections
Anchorage to Foundations Condition of Foundations Overturning Lateral Loads Geologic Site Hazards
Condition of Concrete

Table 10-12 C2, C2A: Concrete Shear Walls with Stiff or Flexible Diaphragms

Typical Deficiencies
Load Path Redundancy Vertical Irregularities Plan Irregularities Lateral Load Path at Pile Caps Deflection Compatibility
Frames Not Part of the Lateral Force Resisting System   Short "Captive" Columns

Cast-in-Place Concrete Shear Walls
   Shear Stress
   Overturning
   Coupling Beams
   Boundary Component Detailing
   Wall Reinforcement

Re-entrant Corners Diaphragm Openings Diaphragm Stiffness/Strength   Sheathing
Diaphragm/Wall Shear Transfer
Anchorage to Foundations Condition of Foundations Overturning Lateral Loads Geologic Site Hazards
Condition of Concrete

Table 10-13 C3, C3A: Concrete Frames with Infill Masonry Shear Walls and Stiff or Flexible Diaphragms

Typical Deficiencies
Load Path Redundancy Vertical Irregularities Plan Irregularities Lateral Load Path at Pile Caps Deflection Compatibility
Frames Not Part of the Lateral Force Resisting System   Complete Frames

Masonry Shear Walls
   Reinforcing in Masonry Walls
   Shear Stress
   Reinforcing at Openings
   Unreinforced Masonry Shear Walls
   Proportions, Solid Walls
   Infill Walls

Re-entrant Corners Diaphragm Openings Diaphragm Stiffness/Strength   Span/Depth Ratio
Diaphragm/Wall Shear Transfer Anchorage for Normal Forces
Anchorage to Foundations Condition of Foundations Overturning Lateral Loads Geologic Site Hazards
Condition of Concrete Quality of Masonry

Table 10-14 PC1: Precast/Tilt-up Concrete Shear Walls with Flexible Diaphragms

Typical Deficiencies
Load Path Redundancy Vertical Irregularities Plan Irregularities Deflection Compatibility
Precast Concrete Shear Walls   Panel-to-Panel Connections
Wall Openings   Collectors

Re-entrant Corners
Crossties
Diaphragm Openings
Diaphragm Stiffness/Strength
  Sheathing
  Unblocked Diaphragms
  Span/Depth Ratio
  Chord Continuity

Diaphragm/Wall Shear Transfer Anchorage for Normal Forces Girder/Wall Connections Stiffness of Wall Anchors
Anchorage to Foundations Condition of Foundation Overturning Lateral Loads Geologic Site Hazards
Condition of Concrete

Table 10-15 PC1A: Precast/Tilt-up Concrete Shear Walls with Stiff Diaphragms

Typical Deficiencies
Load Path Redundancy Vertical Irregularities Plan Irregularities

Precast Concrete Shear Walls
  Panel-to-Panel Connections
  Wall Openings
  Collectors

Re-entrant Corners Diaphragm Openings Diaphragm Stiffness/Strength
Diaphragm/Wall Shear Transfer Anchorage for Normal Forces Girder/Wall Connections
Anchorage to Foundations Condition of Foundations Overturning Lateral Loads Geologic Site Hazards
Condition of Concrete

Table 10-16 PC2: Precast Concrete Frames with Shear Walls

Typical Deficiencies

Load Path
Redundancy
Vertical Irregularities
Plan Irregularities
Lateral Load Path at Pile Caps
Deflection Compatibility

Concrete Moment Frames
  Precast Moment Frame Concerns

Cast-in-Place Concrete Shear Walls
  Shear Stress
  Overturning
  Coupling Beams
  Boundary Component Detailing
  Wall Reinforcement

Re-entrant Corners
Crossties
Diaphragm Openings
Diaphragm Stiffness/Strength

Diaphragm/Wall Shear Transfer
Anchorage for Normal Forces
Girder/Wall Connections
Precast Connections

Anchorage to Foundations
Condition of Foundations
Overturning
Lateral Loads
Geologic Site Hazards

Condition of Concrete

Table 10-17 PC2A: Precast Concrete Frames Without Shear Walls

Typical Deficiencies

Load Path
Redundancy
Vertical Irregularities
Plan Irregularities
Adjacent Buildings
Lateral Load Path at Pile Caps
Deflection Compatibility

Concrete Moment Frames
  Precast Moment Frame Concerns
Frames Not Part of the Lateral Force Resisting System
  Short Captive Columns

Re-entrant Corners
Diaphragm Openings
Diaphragm Stiffness/Strength

Diaphragm/Frame Shear Transfer
Precast Connections

Anchorage to Foundations
Condition of Foundations
Overturning
Lateral Loads
Geologic Site Hazards

Condition of Concrete

Table 10-18 RM1: Reinforced Masonry Bearing Wall Buildings with Flexible Diaphragms

Typical Deficiencies

Load Path
Redundancy
Vertical Irregularities
Plan Irregularities

Masonry Shear Walls
  Reinforcing in Masonry Walls
  Shear Stress
  Reinforcing at Openings

Re-entrant Corners
Crossties
Diaphragm Openings
Diaphragm Stiffness/Strength
  Sheathing
  Unblocked Diaphragms
  Span/Depth Ratio

Diaphragm/Wall Shear Transfer
Anchorage for Normal Forces
Stiffness of Wall Anchors

Anchorage to Foundations
Condition of Foundations
Geologic Site Hazards

Quality of Masonry

Table 10-19 RM2: Reinforced Masonry Bearing Wall Buildings with Stiff Diaphragms

Typical Deficiencies
Load Path Redundancy Vertical Irregularities Plan Irregularities

Masonry Shear Walls
  Reinforcing in Masonry Walls
  Shear Stress
  Reinforcing at Openings

Re-entrant Corners Diaphragm Openings Diaphragm Stiffness/Strength
Diaphragm/Wall Shear Transfer Anchorage for Normal Forces
Anchorage to Foundations Condition of Foundations Geologic Site Hazards
Quality of Masonry

Table 10-20 URM: Unreinforced Masonry Bearing Wall Buildings with Flexible Diaphragms

Typical Deficiencies
Load Path Redundancy Vertical Irregularities Plan Irregularities Adjacent Buildings

Masonry Shear Walls
  Unreinforced Masonry Shear Walls
  Properties, Solid Walls

Re-entrant Corners
Crossties
Diaphragm Openings
Diaphragm Stiffness/Strength
  Sheathing
  Unblocked Diaphragms
  Span/Depth Ratio

Diaphragm/Wall Shear Transfer Anchorage for Normal Forces Stiffness of Wall Anchors
Anchorage to Foundations Condition of Foundations Geologic Site Hazards
Quality of Masonry

Table 10-21 URMA: Unreinforced Masonry Bearing Walls Buildings with Stiff Diaphragms

Typical Deficiencies
Load Path Redundancy Vertical Irregularities Plan Irregularities Adjacent Buildings

Masonry Shear Walls
  Unreinforced Masonry Shear Walls
  Properties, Solid Walls

Re-entrant Corners Diaphragm Openings Diaphragm Stiffness/Strength
Diaphragm/Wall Shear Transfer Anchorage for Normal Forces
Anchorage to Foundations Condition of Foundations Geologic Site Hazards
Quality of Masonry

Table 10-22 Cross Reference Between the Guidelines and FEMA 178 (BSSC, 1992a) Deficiency Reference Numbers

FEMA 178		*Guidelines*	
Section	**Section Heading**	**Section**	**Section Heading**
3.1	**Load Path**	10.3.1.1	Load Path
3.2	**Redundancy**	10.3.1.2	Redundancy
3.3	**Configuration**		
3.3.1	Weak Story	10.3.1.3	Vertical Irregularities
3.3.2	Soft Story	10.3.1.3	Vertical Irregularities
3.3.3	Geometry	10.3.1.3	Vertical Irregularities
3.3.4	Mass	10.3.1.3	Vertical Irregularities
3.3.5	Vertical Discontinuities	10.3.1.3	Vertical Irregularities
3.3.6	Torsion	10.3.1.4	Plan Irregularities
3.4	**Adjacent Buildings**	10.3.1.5	Adjacent Buildings
3.5	**Evaluation of Materials and Conditions**		
3.5.1	Deterioration of Wood	10.3.8.2	Condition of Wood
3.5.2	Overdriven Nails	10.3.8.3	Overdriven Fasteners
3.5.3	Deterioration of Steel	10.3.8.4	Condition of Steel
3.5.4	Deterioration of Concrete	10.3.8.5	Condition of Concrete
3.5.5	Post-Tensioning Anchors	10.3.8.6	Post-Tensioning Anchors
3.5.6	Concrete Wall Cracks	10.3.8.5	Condition of Concrete
3.5.7	Cracks in Boundary Columns	10.3.8.5	Condition of Concrete
3.5.8	Precast Concrete Walls	10.3.8.5	Condition of Concrete
3.5.9	Masonry Joints	10.3.8.7	Quality of Masonry
3.5.10	Masonry Units	10.3.8.7	Quality of Masonry
3.5.11	Cracks in Infill Walls	10.3.8.7	Quality of Masonry
4.1	**Frames with Infill Walls**		
4.1.4	Interfering Walls	10.3.3.3F	Infill Walls
4.2	**Steel Moment Frames**		
4.2.1	Drift Check	10.3.2.1A	Drift
4.2.2	Compact Members	10.3.2.1B	Frames
4.2.3	Beam Penetration	10.3.2.1B	Frames
4.2.4	Moment Connections	10.3.2.1D	Connections
4.2.5	Column Splices	10.3.2.1B	Frames
4.2.6	Joint Webs	10.3.2.1D	Connections
4.2.7	Girder Flange Continuity Plates	10.3.2.1D	Connections
4.2.8	Strong Column-Weak Beam	10.3.2.1C	Strong Column-Weak Beam
4.2.9	Out-of-Plane Bracing	10.3.2.1B	Frames
4.3	**Concrete Moment Frames**		
4.3.1	Shearing Stress Check	10.3.2.2A	Frame and Nonductile Detail Concerns
4.3.2	Drift Check	10.3.2.2A	Frame and Nonductile Detail Concerns

Table 10-22 Cross Reference Between the Guidelines and FEMA 178 (BSSC, 1992a) Deficiency Reference Numbers (continued)

FEMA 178		*Guidelines*	
Section	**Section Heading**	**Section**	**Section Heading**
4.3.3	Prestressed Frame Elements	10.3.2.2A	Frame and Nonductile Detail Concerns
4.3.4	Joint Eccentricity	10.3.2.2A	Frame and Nonductile Detail Concerns
4.3.5	No Shear Failures	10.3.2.2A	Frame and Nonductile Detail Concerns
4.3.6	Strong Column-Weak Beam	10.3.2.2A	Frame and Nonductile Detail Concerns
4.3.7	Stirrup and Tie Hooks	10.3.2.2A	Frame and Nonductile Detail Concerns
4.3.8	Column-Tie Spacing	10.3.2.2A	Frame and Nonductile Detail Concerns
4.3.9	Column-Bar Splices	10.3.2.2A	Frame and Nonductile Detail Concerns
4.3.10	Beam Bars	10.3.2.2A	Frame and Nonductile Detail Concerns
4.3.11	Beam-Bar Splices	10.3.2.2A	Frame and Nonductile Detail Concerns
4.3.12	Stirrup Spacing	10.3.2.2A	Frame and Nonductile Detail Concerns
4.3.13	Beam Truss Bars	10.3.2.2A	Frame and Nonductile Detail Concerns
4.3.14	Joint Reinforcing	10.3.2.2A	Frame and Nonductile Detail Concerns
4.3.15	Flat Slab Frames	10.3.2.2A	Frame and Nonductile Detail Concerns
4.4	**Precast Moment Frames**		
4.4.1	Precast Frames	10.3.2.2B	Precast Moment Frames
4.4.2	Precast Connections	10.3.6.5	Precast Connections
4.5	**Frames Not Part of the Lateral-Force-Resisting System**		
4.5.1	Complete Frames	10.3.2.3A	Complete Frames
5.1	**Concrete Shear Walls**		
5.1.1	Shearing Stress Check	10.3.3.1A	Shearing Stress
5.1.2	Overturning	10.3.3.1B	Overturning
5.1.3	Coupling Beams	10.3.3.1C	Coupling Beams
5.1.4	Column Splices	10.3.3.1D	Boundary Component Detailing
5.1.5	Wall Connection	10.3.3.1D	Boundary Component Detailing
5.1.6	Confinement Reinforcing	10.3.3.1D	Boundary Component Detailing
5.1.7	Reinforcing Steel	10.3.3.1E	Wall Reinforcement
5.1.8	Reinforcing at Openings	10.3.3.1E	Wall Reinforcement
5.2	**Precast Concrete Shear Walls**		
5.2.1	Panel-to-Panel Connections	10.3.3.2A	Panel-to-Panel Connections
5.2.2	Wall Openings	10.3.3.2B	Wall Openings
5.2.3	Collectors	10.3.3.2C	Collectors
5.3	**Reinforced Masonry Shear Walls**		
5.3.1	Shearing Stress Check	10.3.3.3B	Shearing Stress
5.3.2	Reinforcing	10.3.3.3A	Reinforcing in Masonry Walls
5.3.3	Reinforcing at Openings	10.3.3.3C	Reinforcing at Openings

Table 10-22 Cross Reference Between the Guidelines and FEMA 178 (BSSC, 1992a) Deficiency Reference Numbers (continued)

FEMA 178		*Guidelines*	
Section	**Section Heading**	**Section**	**Section Heading**
5.4	**Unreinforced Masonry Shear Walls**		
5.4.1	Shearing Stress Check	10.3.3.3D	Unreinforced Masonry Shear Walls
5.4.2	Masonry Lay-up	10.3.3.3D	Unreinforced Masonry Shear Walls
5.5	**Unreinforced Masonry Infill Walls in Frames**		
5.5.1	Proportions	10.3.3.3E	Proportions of Solid Walls
5.5.2	Solid Walls	10.3.3.3E	Proportions of Solid Walls
5.5.3	Cavity Walls	10.3.3.3F	Infill Walls
5.5.4	Wall Connections	10.3.3.3F	Infill Walls
5.6	**Walls in Wood Frame Buildings**		
5.6.1	Shearing Stress Check	10.3.3.4A	Shear Stress
5.6.2	Openings	10.3.3.4B	Openings
5.6.3	Wall Requirements	10.3.3.4C	Wall Detailing
5.6.4	Cripple Walls	10.3.3.4D	Cripple Walls
6.1	**Concentrically Braced Frames**		
6.1.1	Stress Check	10.3.4.1	System Concerns
6.1.2	Stiffness of Diagonals	10.3.4.2	Stiffness of Diagonals
6.1.3	Tension-Only Braces	10.3.4.2	Stiffness of Diagonals
6.1.4	Chevron Bracing	10.3.4.3	Chevron or K-Bracing
6.1.5	Concentric Joints	10.3.4.4	Braced Frame Connections
6.1.6	Connection Strength	10.3.4.4	Braced Frame Connections
6.1.7	Column Splices	10.3.4.4	Braced Frame Connections
7.1	**Diaphragms**		
7.1.1	Plan Irregularities	10.3.5.1	Re-entrant Corners
7.1.2	Cross Ties	10.3.5.2	Crossties
7.1.3	Reinforcing at Openings	10.3.5.3	Diaphragm Openings
7.1.4	Openings at Shear Walls	10.3.5.3	Diaphragm Openings
7.1.5	Openings at Braced Frames	10.3.5.3	Diaphragm Openings
7.1.6	Openings at Exterior Masonry Shear Walls	10.3.5.3	Diaphragm Openings
7.2	**Wood Diaphragms**		
7.2.1	Sheathing	10.3.5.4A	Board Sheathing
7.2.2	Spans	10.3.5.4C	Spans
7.2.3	Unblocked Diaphragms	10.3.5.4B	Unblocked Diaphragms
7.2.4	Span/Depth Ratio	10.3.5.4D	Span-to-Depth Ratio
7.2.5	Diaphragm Continuity	10.3.5.4E	Diaphragm Continuity
7.2.6	Chord Continuity	10.3.5.4F	Chord Continuity
8.2	**Anchorage for Normal Forces**		
8.2.1	Wood Ledgers	10.3.6.3	Anchorage for Normal Forces

Table 10-22 Cross Reference Between the Guidelines and FEMA 178 (BSSC, 1992a) Deficiency Reference Numbers (continued)

FEMA 178		*Guidelines*	
Section	**Section Heading**	**Section**	**Section Heading**
8.2.2	Wall Anchorage	10.3.6.3	Anchorage for Normal Forces
8.2.3	Masonry Wall Anchors	10.3.6.3	Anchorage for Normal Forces
8.2.4	Anchor Spacing	10.3.6.3	Anchorage for Normal Forces
8.2.5	Tilt-up Walls	10.3.6.3	Anchorage for Normal Forces
8.2.6	Panel-Roof Connection	10.3.6.3	Anchorage for Normal Forces
8.3	**Shear Transfer**		
8.3.1	Transfer to Shear Walls	10.3.6.1	Diaphragm/Wall Shear Transfer
8.3.2	Transfer to Steel Frames	10.3.6.2	Diaphragm/Frame Shear Transfer
8.3.3	Topping Slab to Walls and Frames	10.3.6.1	Diaphragm/Wall Shear Transfer
		10.3.6.2	Diaphragm/Frame Shear Transfer
8.4	**Vertical Components to Foundations**		
8.4.1	Steel Columns	10.3.7.1	Anchorage to Foundations
8.4.2	Concrete Columns	10.3.7.1	Anchorage to Foundations
8.4.3	Wood Posts	10.3.7.1	Anchorage to Foundations
8.4.4	Wall Reinforcing	10.3.7.1	Anchorage to Foundations
8.4.5	Shear-Wall-Boundary Columns	10.3.7.1	Anchorage to Foundations
8.4.6	Wall Panels	10.3.7.1	Anchorage to Foundations
8.4.7	Wood Sills	10.3.7.1	Anchorage to Foundations
8.5	**Interconnection of Elements**		
8.5.1	Girders	10.3.6.4	Girder-Wall Connections
8.5.2	Corbel Bearing	10.3.6.4	Girder-Wall Connections
8.5.3	Corbel Connections	10.3.6.4	Girder-Wall Connections
8.6	**Roof Decking**		
8.6.1	Light-Gage Metal Roof Panels	10.3.6.7	Light Gage Metal, Plastic, or Cementitious Roof Panels
8.6.2	Wall Panels	10.3.6.6	Wall Panels and Cladding
9.1	**Condition of Foundations**		
9.1.1	Foundation Performance	10.3.7.2	Condition of Foundations
9.1.2	Deterioration	10.3.7.2	Condition of Foundations
9.2	**Capacity of Foundations**		
9.2.1	Overturning	10.3.7.3	Overturning
9.2.2	Ties Between Foundation Elements	10.3.7.4	Lateral Loads
9.2.3	Lateral Force on Deep Foundations	10.3.7.4	Lateral Loads
9.2.4	Pole Buildings	10.3.7.4	Lateral Loads
9.2.5	Sloping Sites	10.3.7.4	Lateral Loads

Table 10-22 Cross Reference Between the Guidelines and FEMA 178 (BSSC, 1992a) Deficiency Reference Numbers (continued)

FEMA 178		*Guidelines*	
Section	**Section Heading**	**Section**	**Section Heading**
9.3	**Geologic Site Hazards**		
9.3.1	Liquefaction	10.3.7.5	Geologic Site Hazards
9.3.2	Slope Failure	10.3.7.5	Geologic Site Hazards
9.3.3	Surface Fault Rupture	10.3.7.5	Geologic Site Hazards

11. Architectural, Mechanical, and Electrical Components (Simplified and Systematic Rehabilitation)

11.1 Scope

This chapter establishes rehabilitation criteria for architectural, mechanical, and electrical components and systems that are permanently installed in buildings, or are an integral part of a building system, including their supports and attachments. These components are collectively referred to as "nonstructural components." Contents introduced into buildings by owners or occupants are not within the scope of the *Guidelines.*

Guidance for rehabilitating existing nonstructural components is included within this chapter, while new nonstructural components shall conform to the materials, detailing, and construction requirements for similar elements in new buildings.

Nonstructural components in historic buildings may be highly significant, especially if they are original to the building or innovative for their age. Guidance for their seismic rehabilitation should be sought from the State Historic Preservation Officer or other historic preservation specialist, and from specialized publications. Equally important are other nonseismic considerations, such as accessibility for the disabled, fire protection, and hazardous materials considerations (especially asbestos-containing nonstructural materials). The variety of such nonseismic factors is so great as to make it impossible to treat them in detail in this document.

The assessment process necessary to make a final determination of which nonstructural components are to be rehabilitated is not part of the *Guidelines*, but the subject is touched on briefly in Section 11.3, and the *Commentary* to this chapter provides an outline of an assessment procedure.

The core of this chapter is contained in Table 11-1, which provides:

- A list of nonstructural components subject to Life Safety requirements of these *Guidelines*
- Rehabilitation requirements related to Seismic Zone and Life Safety Performance Level
- Identification of the required Analysis Procedure (analytical or prescriptive)

Section 11.4 provides general requirements and discussion of Rehabilitation Objectives, Performance Levels, and Performance Ranges as they pertain to nonstructural components. Criteria for means of egress are not specifically included in these *Guidelines*; an extensive discussion in the *Commentary* reviews the issues involved if this topic is selected for consideration.

Section 11.5 offers a brief discussion of structural-nonstructural interaction, and Section 11.6 provides general requirements for acceptance criteria for acceleration-sensitive and deformation-sensitive components, and those sensitive to both kinds of response.

Section 11.7 provides sets of equations for a simple "default" force analysis, as well as an extended analysis method that considers a number of additional factors. Another set of equations sets out the Analytical Procedures for determining drift ratios and relative displacements. The general requirements for prescriptive procedures are also set out.

Section 11.8 notes the general ways in which nonstructural rehabilitation is carried out, with a more extended discussion in the *Commentary.*

Sections 11.9, 11.10, and 11.11 provide the rehabilitation criteria for each component category identified in Table 11-1. For each component the following information is given:

- Definition and scope
- Component behavior and rehabilitation concepts
- Acceptance criteria
- Evaluation requirements

Methods of rehabilitation are discussed in more detail in the *Commentary* for each component.

11.2 Procedural Steps

Once the general philosophy of Section 11.1 is understood, its use can be reduced to the following steps, conducted within the framework of a nonstructural hazard mitigation plan (discussed in the *Commentary*, Section C11.3.2).

1. Determine the Performance Level or Range desired.

2. Refer to Table 11-1 to determine for each nonstructural component the applicability of Life Safety or Immediate Occupancy requirements related to seismic zone, and required method of analysis.

3. Refer to Sections 11.9, 11.10, and 11.11 for acceptance criteria for each nonstructural component.

Table 11-1 Nonstructural Components: Applicability of Life Safety and Immediate Occupancy Requirements and Methods of Analysis

COMPONENT		High Seismicity		Moderate Seismicity		Low Seismicity		Analysis Method
		LS	**IO**	**LS**	**IO**	**LS**	**IO**	
A. ARCHITECTURAL								
1.	**Exterior Skin**							
	Adhered Veneer	Yes	Yes	Yes	Yes	No	Yes	F/D
	Anchored Veneer	Yes	Yes	Yes	Yes	No	Yes	F/D
	Glass Blocks	Yes	Yes	Yes	Yes	No	Yes	F/D
	Prefabricated Panels	Yes	Yes	Yes	Yes	Yes	Yes	F/D
	Glazing Systems	Yes	Yes	Yes	Yes	Yes	Yes	F/D
2.	**Partitions**							
	Heavy	Yes	Yes	Yes	Yes	No	Yes	F/D
	Light	No	Yes	No	Yes	No	Yes	F/D
3.	**Interior Veneers**							
	Stone, Including Marble	Yes	Yes	Yes	Yes	No	Yes	F/D
	Ceramic Tile	Yes	Yes	No	Yes	No	Yes	F/D
4.	**Ceilings**							
	a. Directly Applied to Structure	No[13]	Yes	No[13]	Yes	No	Yes	F
	b. Dropped. Furred, Gypsum Board	No	Yes	No	Yes	No	Yes	F
	c. Suspended Lath and Plaster	Yes	Yes	Yes	Yes	No	Yes	F
	d. Suspended Integrated Ceiling	No[11]	Yes	No[11]	Yes	No[11]	Yes	PR
5.	**Parapets and Appendages**	Yes	Yes	Yes	Yes	Yes	Yes	F[1]
6.	**Canopies and Marquees**	Yes	Yes	Yes	Yes	Yes	Yes	F
7.	**Chimneys and Stacks**	Yes	Yes	Yes	Yes	No	Yes	F[2]
8.	**Stairs**	Yes	Yes	Yes	Yes	Yes	Yes	*

Notes and definitions provided on page 11-4

Table 11-1 Nonstructural Components: Applicability of Life Safety and Immediate Occupancy Requirements and Methods of Analysis (continued)

		High Seismicity		Moderate Seismicity		Low Seismicity		
COMPONENT		**LS**	**IO**	**LS**	**IO**	**LS**	**IO**	**Analysis Method**
B. MECHANICAL EQUIPMENT								
1.	**Mechanical Equipment**							
	Boilers and Furnaces	Yes	Yes	Yes	Yes	Yes	Yes	F
	General Mfg. and Process Machinery	No[3]	Yes	No	Yes	No	Yes	F
	HVAC Equipment, Vibration-Isolated	No[3]	Yes	No	Yes	No	Yes	F
	HVAC Equipment, Non-Vibration-Isolated	No[3]	Yes	No	Yes	No	Yes	F
	HVAC Equipment, Mounted In-Line with Ductwork	No[3]	Yes	No	Yes	No	Yes	PR
2.	**Storage Vessels and Water Heaters**							
	Structurally Supported Vessels (Category 1)	No[3]	Yes	No	Yes	No	Yes	Note 4
	Flat Bottom Vessels (Category 2)	No[3]	Yes	No	Yes	No	Yes	Note 5
3.	**Pressure Piping**	Yes	Yes	No	Yes	No	Yes	Note 5
4.	**Fire Suppression Piping**	Yes	Yes	No	Yes	No	Yes	PR
5.	**Fluid Piping, not Fire Suppression**							
	Hazardous Materials	Yes	Yes	Yes	Yes	Yes	Yes	PR/F/D
	Nonhazardous Materials	No	Yes	No	Yes	No	Yes	PR/F/D
6.	**Ductwork**	No[6]	Yes	No[6]	Yes	No	Yes	PR
C. ELECTRICAL AND COMMUNICATIONS								
1.	**Electrical and Communications Equipment**	No[7]	Yes	No[7]	Yes	No	Yes	F
2.	**Electrical and Communications Distribution Equipment**	No[8]	Yes	No[8]	Yes	No	Yes	PR
3.	**Light Fixtures**							
	Recessed	No	No	No	No	No	No	
	Surface Mounted	No	No	No	No	No	No	
	Integrated Ceiling	Yes	Yes	Yes	Yes	No	Yes	PR
	Pendant	No[9]	Yes	No[9]	Yes	No	Yes	F

Notes and definitions provided on page 11-4

Table 11-1 Nonstructural Components: Applicability of Life Safety and Immediate Occupancy Requirements and Methods of Analysis (continued)

COMPONENT		High Seismicity		Moderate Seismicity		Low Seismicity		Analysis Method
		LS	IO	LS	IO	LS	IO	
D. FURNISHINGS AND INTERIOR EQUIPMENT								
1.	**Storage Racks**	Yes[10]	Yes	Yes[10]	Yes	No	Yes	F
2.	**Bookcases**	Yes	Yes	Yes	Yes	No	Yes	F
3.	**Computer Access Floors**	No	Yes	No	Yes	No	Yes	PR/FD
4.	**Hazardous Materials Storage**	Yes	Yes	No[12]	Yes	No[12]	Yes	PR
5.	**Computer and Communication Racks**	No	Yes	No	Yes	No	Yes	PR/F/D
6.	**Elevators**	Yes	Yes	Yes	Yes	No	Yes	F/D
7.	**Conveyors**	No	Yes	No	Yes	No	Yes	F/D

1. Unreinforced masonry parapets not over 4 ft in height may be rehabilitated to Prescriptive Design Concept.
2. Residential masonry chimneys may be rehabilitated to Prescriptive Design Concept.
3. Rehabilitation required to Life Safety Performance Level when:
   Equipment type A or B, or vessel, 6 ft or over in height
   Equipment type C
   Equipment forming part of an emergency power system
   Gas-fired equipment in occupied or unoccupied space
4. Residential water heaters with capacity less than 100 gal may be rehabilitated by Prescriptive Procedure. Other vessels to meet force provisions of Section 11.7.3 or 11.7.4.
5. Vessels or piping systems may be rehabilitated according to Prescriptive Standards. Large systems or vessels shall meet force provisions of Section 11.7.3 or 11.7.4; piping also shall meet drift provisions of Section 11.7.5.
6. Rehabilitation required when ductwork conveys hazardous materials, exceeds 6 sq. ft in cross-sectional area, or is suspended more than 12 in. from top of duct to supporting structure.
7. Rehabilitation required to Life Safety Performance Level when:
   Equipment is 6 ft or over in height
   Equipment weighs over 20 lbs.
   Equipment forms part of an emergency power and/or communication system
8. Rehabilitation required to Life Safety Performance Level when equipment forms part of an emergency lighting, power, and/or communication system
9. Rehabilitation required to Life Safety Performance Level when fixture weight per support exceeds 20 lbs.
10. Rehabilitation not required for storage racks in essentially unoccupied space.
11. Rehabilitation required to Life Safety Performance Level when panels exceed 2 lb/sq. ft and for Enhanced Rehabilitation Objectives.
12. Rehabilitation required where material is in close proximity to occupancy, and leakage can cause immediate life safety threat.
13. Rehabilitation required to achieve Life Safety Performance Level for poorly attached large areas (over 10 sq. ft) of plaster ceilings on metal or wood lath.

Key:

LS Life Safety Performance Level
IO Immediate Occupancy Performance Level
PR Prescriptive Procedure acceptable
F Analytical Procedure: force analysis, Section 11.7.3 or 11.7.4
F/D Analytical Procedure: force and relative displacement analysis, Sections 11.7.4 and 11.7.5
* Rehabilitate as required for individual components

4. Use equations in Section 11.7 to conduct any necessary analysis.
5. Develop any necessary design solutions to meet the force requirements, the deformation criteria, and any prescriptive requirements. For capacities of

nonstructural components and their connections refer to Chapters 5 through 8, or derive capacity values in a manner consistent with those chapters.

11.3 Historical and Component Evaluation Considerations

11.3.1 Historical Perspective

Prior to the 1961 *Uniform Building Code* and the 1964 Alaska earthquake, architectural components and mechanical and electrical systems for buildings had typically been designed with little, if any, regard to stability when subjected to seismic forces. By the time of the 1971 San Fernando earthquake, it became quite clear that damage to nonstructural elements could result in serious casualties, severe building functional impairment, and major economic losses even when the structural damage was not significant (Lagorio, 1990).

The architectural, mechanical, and electrical components and systems of a historic building may be very significant, especially if they are original to the building, very old, or innovative. An assessment of their significance by an appropriate professional—such as an architectural historian, historical preservation architect, or historian of engineering and technology—may be necessary. Historic buildings may also have materials, such as lead pipes and asbestos, that may or may not pose a hazard depending on their location, condition, use or abandonment, containment, and/or disturbance during the rehabilitation.

Readers are referred to the *Commentary* to this section for further discussion and a chronology of the introduction of nonstructural considerations into seismic codes.

11.3.2 Component Evaluation

Procedures for detailed assessment to decide which existing nonstructural components should be rehabilitated are not part of these *Guidelines*. However, there is a brief discussion under “Evaluation Needs” in each component section. To achieve the Basic Safety Objective (BSO), nonstructural components as listed in Table 11-1 must meet the Life Safety Performance Level for specified ground motion, as defined in Chapter 2. In other cases—such as when the Limited Safety Performance Range applies—there may be more latitude in the selection of components for rehabilitation. A suggested procedure for the detailed evaluation of existing nonstructural components—with cost-effectiveness and a ranking of importance in mind—is outlined in the *Commentary*, Section C11.3.2.

11.4 Rehabilitation Objectives, Performance Levels, and Performance Ranges

The nonstructural Rehabilitation Objective may be the same as for the structural rehabilitation, or may differ, except for the BSO, in which case structural and nonstructural requirements specified in the *Guidelines* must be met.

These *Guidelines* are also intended to be applicable to the situation where nonstructural—but not structural—components are to be rehabilitated. Rehabilitation that is restricted to the nonstructural components will typically fall within the Limited Safety Performance Range, unless the structure is already determined to meet a specified Rehabilitation Objective.

To qualify for any Rehabilitation Objective higher than Limited Safety, consideration of structural behavior is necessary even if only nonstructural components are to be rehabilitated, to properly take into account loads on nonstructural components generated by inertial forces or imposed deformations.

11.4.1 Performance Levels for Nonstructural Components

Four Nonstructural Performance Levels and three Structural Performance Levels are described in Chapter 2 of the *Guidelines*. For nonstructural components, the Collapse Prevention Performance Level does not, in general, apply, since most nonstructural damage resulting from a building at the Collapse Prevention damage state is regarded as acceptable. (Rehabilitation of parapets and heavy appendages is required, however, for conformance with the Collapse Prevention Building Performance Level.) The four defined Performance Levels applying to nonstructural components are:

- **Hazards Reduced Performance Level.** This represents a post-earthquake damage level in which extensive damage has occurred to nonstructural components but large or heavy items—such as parapets, cladding, plaster ceilings, or storage racks—posing a falling hazard to many people are prevented from falling.

- **Life Safety Performance Level**. This Performance Level is intended primarily to prevent nonstructural falling hazards that can directly cause injury. Excluded from the Life Safety Performance Level are specific criteria relating to post-earthquake nonstructural performance, such as egress, alarm and communications systems, fire protection systems, and other functional issues. The issue of egress protection, although not specifically addressed, is substantially taken care of by rehabilitation of relevant nonstructural components to the Life Safety Performance Level.

  Acceptance Criteria for the Life Safety Performance Level are provided in the sections on each nonstructural equipment category.

  Post-earthquake functional concerns are addressed within the Damage Control Performance Range and by the Immediate Occupancy Performance Level.

- **Immediate Occupancy Performance Level.** To attain this Performance Level, conformance with requirements for the Life Safety Performance Level must be met, together with the requirements for Immediate Occupancy where applicable.

Acceptance criteria for the Immediate Occupancy Performance Level are provided only in the sections on each nonstructural component category.

- **Operational Performance Level.** A theoretical Building Performance Level beyond Immediate Occupancy, this level depends on the continuing functioning of all utilities and systems, and, often, of other sensitive equipment. Specific criteria for nonstructural components for this Performance Level are not provided in these *Guidelines* because the critical components and systems are building-specific, and operational capability may be dependent on equipment over which the design team has no authority.

The procedure for attaining an Operational Performance Level is to use the criteria for Immediate Occupancy and develop additional criteria based on a detailed evaluation of the specific building relative to the operational functions to be maintained.

Tables 2-6 through 2-8 summarize nonstructural damage states in relation to Performance Levels.

11.4.2 Performance Ranges for Nonstructural Components

Including the Hazards Reduced Performance Level, below the Life Safety Nonstructural Performance Level, there are nonstructural rehabilitation damage states that will fall below or above the Life Safety Level. For example, it is possible to exceed the Life Safety Level but fall short of Immediate Occupancy, or exceed Immediate Occupancy but not meet Operational Performance Level requirements. Performance in excess of the Operational Performance Level is also conceivable, though unlikely. While the ranges may be conceptually referred to as Enhanced or Limited (relative to Life Safety), such ranges are not formally defined by the *Guidelines* for nonstructural components, nor are requirements specified.

11.4.3 Regional Seismicity and Nonstructural Components

Requirements for the rehabilitation of nonstructural components relating to the three Seismic Zones—High, Moderate, and Low—are shown in Table 11-1 and noted in each section, where applicable. In general, in regions of low seismicity, certain nonstructural components have no rehabilitation requirements with respect to the Life Safety Performance Level. Rehabilitation of these components, particularly where rehabilitation is simple, may nevertheless be desirable for damage control and property loss reduction.

11.4.4 Means of Egress: Escape and Rescue

Emergency post-earthquake access into and out of buildings is one of the aspects of nonstructural performance that may be selected for consideration in the Damage Control Performance Range. Because the Damage Control Performance Range is not specifically defined by requirements in the *Guidelines*, emergency escape and rescue criteria are not included within the *Guidelines*.

Preservation of egress is accomplished primarily by ensuring that the most hazardous nonstructural elements are replaced or rehabilitated. The items listed in Table 11-1 for achieving the Life Safety Performance Level show that typical requirements for maintaining egress will, in effect, be accomplished if the egress-related components are addressed. These would include the following items listed in FEMA 178, *NEHRP Handbook for the Seismic Evaluation of Existing Buildings* (pp. 91–92, and pp. A-20) (BSSC, 1992b).

- Walls around stairs, elevator enclosures, and corridors are not hollow clay tile or unreinforced masonry.
- Stair enclosures do not contain any piping or equipment except as required for life safety.
- Veneers, cornices, and other ornamentation above building exits are well anchored to the structural system.
- Parapets and canopies are anchored and braced to prevent collapse and blockage of building exits.

Beyond this, the following list describes some conditions that might be commonly recognized as representing major obstruction; the building should be inspected to see whether these, or any similar hazardous conditions exist; if so, their replacement or rehabilitation should be included in the rehabilitation plan.

- Partitions taller than six feet and weighing more than five pounds per square foot, if collapse of the entire partition—rather than cracking—is the expected mode of failure, and if egress would be impeded
- Ceilings, soffits, or any ceiling or decorative ceiling component weighing more than two pounds per square foot, if it is expected that large areas (pieces measuring ten square feet or larger) would fall
- Potential for falling ceiling-located light fixtures or piping; diffusers and ductwork, speakers and alarms, and other objects located higher than 42 inches off the floor
- Potential for falling debris weighing more than 100 pounds that, if it fell in an earthquake, would obstruct a required exit door or other component, such as a rescue window or fire escape
- Potential for jammed doors or windows required as part of an exit path—including doors to individual offices, rest rooms, and other occupied spaces

Of these, the first four are also taken care of in the Life Safety Performance Level requirement. The last condition is very difficult to remove with any assurance, except for low levels of shaking in which structural drift and deformation will be minimal, and the need for escape and rescue correspondingly slight.

Refer to the *Commentary* for this section for further discussion of egress, escape, and rescue issues.

11.5 Structural-Nonstructural Interaction

11.5.1 Response Modification

In cases where a nonstructural component directly modifies the strength or stiffness of the building structural elements, or its mass affects the building loads, its characteristics should be considered in the structural analysis of the building. Particular care should be taken to identify masonry infill that could reduce the effective length of adjacent columns.

11.5.2 Base Isolation

Nonstructural components that cross the isolation interface in a base-isolated structure should be designed to accommodate the total maximum displacement of the isolator.

11.6 Acceptance Criteria for Acceleration-Sensitive and Deformation-Sensitive Components

11.6.1 Acceleration-Sensitive Components

Acceleration-sensitive components shall meet the force requirements derived from equations in Section 11.7. Acceleration-sensitive components are discussed, where necessary, in each component section (Sections 11.9, 11.10, and 11.11). The guiding principle for deciding whether a component requires a force analysis, as defined in Section 11.7, is that analysis of inertial loads generated within the component is necessary to properly consider the component's seismic behavior. The steps for application of acceleration-sensitive acceptance criteria are as follows:

1. Determination of the Rehabilitation Objective and associated Performance Level (see Table 11-1 for

the applicability of requirements keyed to the Life Safety Performance Level)

2. Determination of the seismicity—Low, Moderate, or High—as defined in Section 2.6.3

3. Application of design forces to the existing or modified component (Section 11.7), if the Analytical Procedure is required by Table 11-1; or, if the Prescriptive Procedure is acceptable according to Table 11-1, comparison of the existing component with required characteristics as defined in a reference or standard

4. Verification that the component can meet the acceptance criteria for the applicable Performance Level (see each specific component section, Sections 11.9, 11.10, and 11.11).

11.6.2 Deformation-Sensitive Components

Deformation-sensitive components shall meet the general acceptance criteria of this section, as well as additional requirements listed for specific components. The steps for application of deformation-sensitive acceptance criteria are:

1. Determination of the Rehabilitation Objective and associated Performance Level (see Table 11-1 for the applicability of requirements keyed to Performance Level) shall be made.

2. Determination of the seismicity—Low, Moderate, or High—as defined in Section 2.6.3, shall be made.

3. Determination of the deformation and associated drift ratio of the structural component(s) to which the deformation-sensitive nonstructural component is attached (see structural Analysis Procedures of preceding sections) shall be made.

4. Analysis shall be made of the nonstructural component's response to the deformation of the structure, including a consideration of the transfer of loads through the particular connection details of the nonstructural component, or comparison of the existing component with required characteristics as defined in a reference or standard, if the Prescriptive Procedure is acceptable according to Table 11-1.

5. Verification shall be made that the component can meet the acceptance criteria for the applicable Performance Level (see each specific component section, Sections 11.9, 11.10 and 11.11). In lieu of application of the specific acceptance criteria listed for each component, the following requirements may be used:

Life Safety Performance Level. The component meets deformation-sensitive acceptance criteria if the drift ratio at that story level is 0.01 or less. (This alternative will require consideration of glazing or other components that can hazardously fail at lesser drift ratios—depending on installation details—or components that can undergo greater distortion without hazardous failure resulting—for example, typical gypsum board partitions. This alternative may be appropriate only where the Prescriptive Procedure is allowed [though calculations are required here because the structure's drift must be known].)

Use of Drift Ratio Values as Acceptance Criteria. The data on drift ratio values related to damage states is limited, and the use of single median drift ratio values as acceptance criteria must cover a broad range of actual conditions. It is therefore suggested that the limiting drift values shown in this chapter be used as a guide for evaluating the probability of a given damage state for a subject building, but not be used as absolute acceptance criteria. At higher Performance Levels it is likely that the criteria for nonstructural deformation-sensitive components may control the structural rehabilitation design. These criteria should be regarded as a flag for the careful evaluation of structural-nonstructural interaction and consequent damage states, rather than the required imposition of an absolute acceptance criterion that might require costly redesign of the structural rehabilitation. For further discussion, see the *Commentary* for this section.

11.6.3 Acceleration- and Deformation-Sensitive Components

Some components are both acceleration- and deformation-sensitive. They must be analyzed for conformance to acceptance criteria for both forms of response.

11.7 Analytical and Prescriptive Procedures

11.7.1 Application of Analytical and Prescriptive Procedures

There are two nonstructural rehabilitation procedures:

- Prescriptive Procedure
- Analytical Procedure

There are three analysis methods for calculating forces within the Analytical Procedure.

- Equation 11-1, a simple conservative default equation, may be used.
- Equations 11-2 and 11-3 offer more complete equivalent lateral force equations. In addition, Equations 11-4 and 11-5 should be used when drift is a consideration.
- Results from any structural analysis method allowed for the building's rehabilitation may be used, as long as Performance Level criteria, response modification factors, and other considerations are treated consistently. The Analytical Procedure is always acceptable; the Prescriptive Procedure is acceptable for the combinations of seismicity, Performance Level, and component listed in Table 11-1.

11.7.2 Prescriptive Procedure

A Prescriptive Procedure consists of published standards and references that describe the design concepts and construction features that must be present for a given nonstructural component to be seismically protected. No engineering calculations are required in a Prescriptive Procedure, although in some cases an engineering review of the design and installation is required.

Where a Prescriptive Procedure is allowed, the specific prescriptive references are given in the section on the individual component, Sections 11.9, 11.10, and 11.11.

11.7.3 Analytical Procedure: Default Equation

Seismic forces shall be determined in accordance with Equation 11-1:

$$F_p = 1.6\, S_{XS} I_p W_p \qquad (11\text{-}1)$$

where

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

S_{XS} = Spectral response acceleration at short periods for any hazard level

I_p = Component performance factor that is either 1.0 for Life Safety Performance Level or 1.5 for Immediate Occupancy Performance Level

W_p = Component operating weight

11.7.4 Analytical Procedure: General Equation

Alternatively, seismic forces shall be determined in accordance with Equations 11-2 and 11-3

$$F_p = \frac{0.4 a_p S_{XS} I_p W_p \left(1 + \frac{2x}{h}\right)}{R_p} \qquad (11\text{-}2)$$

Note: F_p calculated from Equation 11-2 need not exceed F_p calculated from Equation 11-1.

$$F_p \text{ (minimum)} = 0.3\, S_{XS}\, I_p\, W_p \qquad (11\text{-}3)$$

where

a_p = Component amplification factor, related to rigidity of component that varies from 1.00 to 2.50 (select appropriate value from Table 11-2)

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

S_{XS} = Spectral response acceleration at short periods for any hazard level

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

I_p = Component performance factor that is either 1.0 for Life Safety Performance Level or 1.5 for Immediate Occupancy Performance Level

R_p = Component response modification factor, related to ductility of anchorage that varies from 1.25 to 6.0 (select appropriate value from Table 11-2)

W_p = Component operating weight

x = Elevation in structure of component relative to grade elevation

11.7.5 Drift Ratios and Relative Displacements

Drift ratios (D_r) shall be determined in accordance with the following equations:

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

$$D_r = (\delta_{xA} - \delta_{yA}) / (X - Y) \qquad (11\text{-}4)$$

Relative displacements (D_p) shall be determined in accordance with the following equation:

For relative displacement of two connection points on separate buildings or structural systems, use

$$D_p = |\delta_{xA}| + |\delta_{xB}| \qquad (11\text{-}5)$$

where

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

D_r = Drift ratio

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

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

δ_{xA} = Deflection at building level x of Building A, determined by analysis as defined in Chapter 3

δ_{yA} = Deflection at building level y of Building A, determined by analysis as defined in Chapter 3

δ_{xB} = Deflection at building level x of Building B, determined by analysis as defined in Chapter 3

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

11.7.6 Other Procedures

Other procedures are available that require determination of the maximum acceleration of the building at each component support and the maximum relative displacements between supports common to an individual component.

Linear Procedures can be used to calculate the maximum acceleration of each component support and the inter-story drifts of the building, taking into account the location of the component in the building. Consideration of the flexibility of the component, and the possible amplification of the building roof and floor accelerations and displacements in the component, would require the development of roof and floor response spectra or acceleration time histories at the nonstructural support locations, derived from the dynamic response of the structure.

Relative displacements between component supports are difficult to calculate, even with the use of acceleration time histories, because the maximum displacement of each component support at different levels in the building might not occur at the same time during the building response.

Guidelines for these dynamic analyses for nonstructural components are given in Chapter 6 of *Seismic Design Guidelines for Essential Buildings*, a supplement to *Seismic Design of Buildings* (Department of the Army, Navy, and Air Force, 1986).

These other analytical procedures are considered too complex for the rehabilitation of nonessential building nonstructural components for Immediate Occupancy and Life Safety Performance Levels.

Recent research (Drake and Bachman, 1995) has shown that the Analytical Procedures in Sections 11.7.3 and 11.7.4, which are based on the 1997 *NEHRP Provisions for New Buildings* (BSSC, 1997) Analytical Procedures, provide a reasonable upper bound for the seismic forces on nonstructural components.

Therefore, the other complex analytical procedures outlined above to develop roof and floor spectra are not required to evaluate and rehabilitate the typical nonstructural components discussed in this chapter. Use of the Analytical Procedures in Sections 11.7.3 and 11.7.4 is recommended.

Table 11-2 Nonstructural Component Amplification and Response Modification Factors

COMPONENT		a_p[1]	R_p[2]
A.	**ARCHITECTURAL**		
1.	**Exterior Skin**		
	Adhered Veneer	1	4
	Anchored Veneer	1	3[3]
	Glass Block	1	2
	Prefabricated Panels	1	3[3]
	Glazing Systems	1	2
2.	**Partitions**		
	Heavy	1	1.5
	Light	1	3
3.	**Interior Veneers**		
	Stone, Including Marble	1	1.5
	Ceramic Tile	1	1.5
4.	**Ceilings**		
	a. Directly Applied to Structure	1	1.5
	b. Dropped, Furred Gypsum Board	1	1.5
	c. Suspended Lath and Plaster	1	1.5
	d. Suspended Integrated Ceiling	1	1.5
5.	**Parapets and Appendages**	2.5	1.25
6.	**Canopies and Marquees**	2.5	1.5
7.	**Chimneys and Stacks**	2.5	1.25
8.	**Stairs**	1	3
B.	**MECHANICAL EQUIPMENT**		
1.	**Mechanical Equipment**		
	Boilers and Furnaces	1	3
	General Mfg. and Process Machinery	1	3
	HVAC Equipment, Vibration-Isolated	2.5	3
	HVAC Equipment, Non-Vibration-Isolated	1	3
	HVAC Equipment, Mounted In-Line with Ductwork	1	3
2.	**Storage Vessels and Water Heaters**		
	Vessels on Legs (Category 1)	2.5	1.5
	Flat Bottom Vessels (Category 2)	2.5	3
3.	**High-Pressure Piping**	2.5	4
4.	**Fire Suppression Piping**	2.5	4

Table 11-2 Nonstructural Component Amplification and Response Modification Factors (continued)

COMPONENT		a_p[1]	R_p[2]
5.	**Fluid Piping, not Fire Suppression**		
	Hazardous Materials	2.5	1
	Nonhazardous Materials	2.5	4
6	**Ductwork**	1	3
C.	**ELECTRICAL AND COMMUNICATIONS EQUIPMENT**		
1.	**Electrical and Communications Equipment**	1	3
2.	**Electrical and Communications Distribution Equipment**	2.5	5
3.	**Light Fixtures**		
	Recessed	1	1.5
	Surface Mounted	1	1.5
	Integrated Ceiling	1	1.5
	Pendant	1	1.5
D.	**FURNISHINGS AND INTERIOR EQUIPMENT**		
1.	**Storage Racks**[4]	2.5	4
2.	**Bookcases**	1	3
3.	**Computer Access Floors**	1	3
4.	**Hazardous Materials Storage**	2.5	1
5.	**Computer and Communications Racks**	2.5	6
6.	**Elevators**	1	3
7.	**Conveyors**	2.5	3

1. A lower value for a_p may be justified by detailed dynamic analysis. The value for a_p shall be not less than 1. The value of $a_p = 1$ is for equipment generally regarded as rigid and rigidly attached. The value of $a_p = 2.5$ is for equipment generally regarded as flexible and flexibly attached. See the definitions (Section 11.12) for explanations of "Component, rigid" and "Component, flexible." Where flexible diaphragms provide lateral support for walls and partitions, the value of a_p shall be increased to 2.0 for the center one-half of the span.
2. $R_p = 1.5$ for anchorage design where component anchorage is provided by expansion anchor bolts, shallow chemical anchors, or shallow (nonductile) cast-in-place anchors, or where the component is constructed of nonductile materials. Shallow anchors are those with an embedment length-to-bolt diameter ratio of less than eight.
3. Applies when attachment is ductile material and design, otherwise 1.5.
4. Storage racks over six feet in height shall be designed in accordance with the provisions of Section 11.11.1.

11.8 Rehabilitation Concepts

Nonstructural rehabilitation is accomplished through replacement, strengthening, repair, bracing, or attachment. These methods are discussed in more depth in the *Commentary* to this section.

11.9 Architectural Components: Definition, Behavior, and Acceptance Criteria

11.9.1 Exterior Wall Elements

11.9.1.1 Adhered Veneer

A. Definition and Scope

Adhered veneer includes thin exterior finish materials secured to a backing material by adhesives. The backing may be masonry, concrete, cement plaster, or a structural framework material. The four main categories of adhered veneer are:

1. Tile, masonry, stone, terra cotta, or other similar materials not over one inch thick
2. Glass mosaic units not over 2" x 2" x 3/8" thick
3. Ceramic tile
4. Exterior plaster (stucco)

B. Component Behavior and Rehabilitation Concepts

Adhered veneers are predominantly deformation-sensitive; deformation of the substrate leads to cracking or separation of the veneer from its backing. Poorly adhered veneers may be dislodged by direct acceleration.

Calculation of the drift of the structure to which the nonstructural component is attached is necessary to establish conformance with drift acceptance criteria related to Performance Level. Nonconformance requires limiting drift, special detailing to isolate substrate from structure to permit drift, or replacement with drift-tolerant material. Poorly adhered veneers should be replaced.

C. Acceptance Criteria

Life Safety Performance Level. Compliance is provided by design of the attachment to the backing to meet the out-of-plane force provisions of Section 11.7.3 or 11.7.4 and to meet the relative displacement in-plane drift provisions of Section 11.7.5. The limiting in-plane drift ratio is 0.03.

Immediate Occupancy Performance Level. Compliance is provided by design of the attachment to the backing to meet the out-of-plane force provisions of Section 11.7.3 or 11.7.4 and to meet the relative displacement in-plane drift provisions of Section 11.7.5. The limiting in-plane drift ratio is 0.01.

D. Evaluation Requirements

Adhered veneer must be evaluated by visual observation, as well as tapping to discern looseness or cracking that may be present. If found, this may indicate either defective bonding to the substrate or excessive flexibility of the supporting structure.

11.9.1.2 Anchored Veneer

A. Definition and Scope

Anchored veneer includes masonry or stone units that are attached to the supporting structure by mechanical means. The three main categories of anchored veneer are:

1. Masonry and stone units not over five inches nominal thickness
2. Stone units from five inches to ten inches nominal thickness
3. Stone slab units not over two inches nominal thickness

The provisions of this section apply to units that are more than 48 inches above the ground or adjacent exterior area.

B. Component Behavior and Rehabilitation Concepts

Anchored veneer is both acceleration- and deformation-sensitive. Heavy units may be dislodged by direct acceleration, which distorts or fractures the mechanical connections. Deformation of the supporting structure, particularly if it is a frame, may similarly affect the connections, and the units may be displaced or dislodged by racking.

Drift analysis is necessary to establish conformance with drift acceptance criteria related to Performance Level. Nonconformance requires limiting structural drift, or special detailing to isolate substrate from structure to permit drift. Defective connections must be replaced.

C. Acceptance Criteria

Life Safety Performance Level. Compliance is provided by design of the attachment to the backing to meet the out-of-plane force provisions of Section 11.7.3 or 11.7.4 and relative displacement to meet the relative displacement in-plane drift provisions of Section 11.7.5. The limiting drift ratio is 0.02.

Immediate Occupancy Performance Level. Compliance is provided by design of the attachment to the backing to meet the out-of-plane force provisions of Section 11.7.3 or 11.7.4 and to meet the relative displacement in-plane drift provisions of Section 11.7.5. The limiting drift ratio is 0.01.

D. Evaluation Requirements

The stone units must have adequate stability, joint detailing, and maintenance to prevent moisture penetration from weather that could destroy the anchors. The anchors must be visually evaluated and, based on the engineer's judgment, tested to establish capacity to sustain design forces and deformations.

11.9.1.3 Glass Block Units and Other Nonstructural Masonry

A. Definition and Scope

This category includes glass block, and other units that are self-supporting for static vertical loads, held together by mortar, and structurally detached from the surrounding structure.

B. Component Behavior and Rehabilitation Concepts

These units are both acceleration- and deformation-sensitive; failure in-plane generally occurs by deformation in the surrounding structure that results in unit cracking and displacement along the cracks. Failure out-of-plane takes the form of dislodgment or collapse caused by direct acceleration.

For small wall areas (less than 144 square feet or 15 feet in any dimension), rehabilitation can be accomplished by restoration, using the Prescriptive Procedure based on the *Uniform Building Code*, 1994, Section 2110 (ICBO, 1994). For larger areas, the Analytical Procedure must be used to establish forces and drifts against which the design must be measured. Nonconformance with deformation criteria requires limiting structural drift, or special detailing to isolate the glass block wall from the surrounding structure to permit drift. Sufficient reinforcing must be provided to deal with out-of-plane forces. Large walls may need to be subdivided by additional structural supports into smaller areas that can meet the drift or force acceptance criteria.

C. Acceptance Criteria

Life Safety Performance Level. Compliance is provided by design of the glass block wall and its enclosing framing, to meet both the in-plane and out-of-plane force provisions of Section 11.7.3 or 11.7.4 and to meet the relative displacement in-plane drift provisions of Section 11.7.5. The limiting drift ratio is 0.02.

Immediate Occupancy Performance Level . Compliance is provided by design of the glass block wall and its enclosing framing, to meet both the in-plane and out-of-plane force provisions of Section 11.7.3 or 11.7.4 and to meet the relative displacement in-plane drift provisions of Section 11.7.5. The limiting drift ratio is 0.01.

D. Evaluation Requirements

The Prescriptive Procedure referred to above will serve as the criteria against which the wall must be evaluated.

11.9.1.4 Prefabricated Panels

A. Definition and Scope

This category consists of prefabricated panels that are installed with adequate structural strength within themselves and their connections to resist wind, seismic, and other forces. These panels are generally attached around their perimeters to the primary structural system. The three typical types of prefabricated panels are the following:

1. Precast concrete, and concrete panels with facing (generally stone) laminated or mechanically attached
2. Laminated metal-faced insulated panels
3. Steel strong-back panels, with insulated, water-resistant facing, or mechanically attached metal or stone facing

B. Component Behavior and Rehabilitation Concepts

Prefabricated panels are both acceleration- and deformation-sensitive. Lightweight units may be damaged by racking; heavy units may be dislodged by direct acceleration, which distorts or fractures the mechanical connections. Excessive deformation of the supporting structure—most likely if it is a frame—may result in the units imposing external racking forces on one another, and distorting or fracturing their connections, with consequent displacement or dislodgment.

Drift analysis is necessary to establish conformance with drift acceptance criteria related to Performance Level. Nonconformance requires limiting structural drift, or special detailing to isolate panels from structure to permit drift; this generally requires panel removal. Defective connections must be replaced.

C. Acceptance Criteria

Life Safety Performance Level. Compliance is provided by design of the panel and connections to meet the in-plane and out-of-plane force provisions of Section 11.7.3 or 11.7.4 and to meet the relative displacement in-plane drift provisions of Section 11.7.5. The limiting drift ratio is 0.02.

Immediate Occupancy Performance Level. Compliance is provided by design of the panel and connections to meet the in-plane and out-of-plane force provisions of Section 11.7.3 or 11.7.4 and to meet the relative displacement in-plane drift provisions of Section 11.7.5. The limiting drift ratio is 0.01.

D. Evaluation Requirements

The attachment of prefabricated panels to the structure must be evaluated for in- and out-of-plane forces and for in-plane displacement. Connections must be visually inspected and, based on the engineer's judgment, testing to establish capacity to sustain design forces and loads.

11.9.1.5 Glazing Systems

A. Definition and Scope

Glazing systems consist of assemblies of walls that are made up from structural subframes attached to the main structure. The subframes may be assembled in the field, or prefabricated in sections and assembled in the field. Five typical categories of glazing system are:

1. Stick curtainwall systems, assembled on site
2. Unitized curtain wall systems, assembled from prefabricated units
3. Sloped glazing and skylights—may be prefabricated units or assembled on site
4. "Storefront" type glazing, assembled on site
5. Structural glazing in which the glass is attached to its supporting framework on two or four sides with adhesive silicone without mechanical restraint

Within each of these categories, there are three basic types of glazed openings:

1. Marine glazing (mostly factory built), in which the glass is clasped in a "U" rubber or vinyl gasket and then surrounded by a screwed-together aluminum frame (i.e., sliding doors and windows)
2. "Wet" glazing, in which the glass is held into the frame with silicone or other sealant compound or is attached to the frame with silicone as in structural glazing
3. "Dry" glazing, in which the glass is held into the frame with either putty, a rubber/vinyl bead, or wood/metal stops

B. Component Behavior and Rehabilitation Concepts

Glazing systems are predominantly deformation-sensitive, but may also become displaced or detached by large acceleration forces. Failures predominantly stem from the third method of glazing ("dry" glazing), and generally occur by the glass shattering due to in-plane displacements, or glass falling out of its supporting frame due to out-of-plane forces, often combined with loss of edge blocks and sealant strips caused by racking.

Drift analysis is necessary to establish conformance with drift acceptance criteria related to Performance Level. Nonconformance requires limiting structural drift, or special detailing to isolate the glazing system from the structure to permit drift; this would require removal of the glazing system and replacement with an alternative design. Glazing with insufficient edge bite or insufficient resilience and clearance from the metal framing must be reglazed.

C. Acceptance Criteria

Life Safety Performance Level. Compliance is provided by design of the glazing system and its supporting structure to meet the force provisions of Section 11.7.3 or 11.7.4 for out-of-plane forces, and to meet the relative displacement in-plane drift provisions of Section 11.7.5. The limiting drift ratio is 0.02.

Immediate Occupancy Performance Level. Compliance is provided by design of the glazing system and its supporting structure to meet the force provisions of Section 11.7.3 or 11.7.4 for out-of-plane forces, and to meet the relative displacement in-plane drift provisions of Section 11.7.5. The limiting drift ratio is 0.01.

D. Evaluation Requirements

Glazed walls must be evaluated visually to determine the details of glass support, mullion configuration, sealant (wet or dry), and connectors.

11.9.2 Partitions

11.9.2.1 Definition and Scope

Partitions are vertical non-load-bearing interior elements that provide space division. They may span laterally from floor to underside of floor or roof above, with connections at the top that may or may not allow for isolation from in-plane drift. Other partitions extend only up to a hung ceiling, and may or may not have lateral bracing above that level to structural support, or may be freestanding.

Heavy partitions are constructed of masonry materials such as hollow clay tile or concrete block, or are assemblies that weigh five pounds per square foot or more.

Light partitions are constructed of metal or wood studs surfaced with lath and plaster, gypsum board, wood, or other facing materials, and weigh less than five pounds per square foot.

Glazed partitions that span from floor to ceiling or to the underside of floor or roof above are subject to the requirements of Section 11.9.1.5.

Modular office furnishings that include movable partitions are considered as contents rather than partitions, and as such are not within the *Guidelines*' scope.

Heavy partitions—whether infill or freestanding—constructed of masonry materials, such as hollow clay tile or concrete block, are subject to the requirements of Chapter 7.

11.9.2.2 Component Behavior and Rehabilitation Concepts

Partitions are both acceleration- and deformation-sensitive. Partitions attached to the structural floors both above and below, and loaded in-plane, can experience shear cracking, distortion and fracture of the partition framing, and detachment of the surface finish, because of structural deformations. Similar partitions loaded out-of-plane can experience flexural cracking, failure of connections to structure, and collapse. The high incidence of unsupported block partitions in low and moderate seismic zones represents a significant collapse threat.

Partitions subject to deformations from the structure can be protected by providing a continuous gap between the partition and the surrounding structure, combined with attachment that provides for in-plane movement but out-of-plane restraint. Lightweight partitions that are not part of a fire-resistive system are regarded as replaceable. Refer to the *Commentary* for discussion on rehabilitation of lightweight partitions used as fire walls.

11.9.2.3 Acceptance Criteria

Life Safety Performance Level

Heavy Partitions. Compliance is provided by design of the partitions to meet the out-of-plane force provisions of Section 11.7.3 or 11.7.4 and to meet the in-plane relative displacement provisions of Section 11.7.5. The limiting drift ratio is 0.01.

Light Partitions. No requirements.

Immediate Occupancy Performance Level

Heavy Partitions. Compliance is provided by design of the partitions to meet the out-of-plane force provisions of Section 11.7.3 or 11.7.4 and to meet the in-plane relative displacement drift provisions of Section 11.7.5. The limiting drift ratio is 0.005.

Light Partitions. Compliance is provided by design of the partitions to meet the out-of-plane force provisions of Section 11.7.3 or 11.7.4 and to meet the in-plane relative displacement drift provisions of Section 11.7.5. The limiting drift ratio is 0.01.

11.9.2.4 Evaluation Requirements

Partitions must be evaluated to ascertain the type of material. For concrete block partitions, presence of reinforcing and connection conditions at edges are important. For light partitions, bracing, or anchoring of the top of the partitions, is important.

11.9.3 Interior Veneers

11.9.3.1 Definition and Scope

Interior veneers are thin decorative-finish materials applied to interior walls and partitions. These provisions apply to veneers mounted four feet or more above the floor.

11.9.3.2 Component Behavior and Rehabilitation Concepts

Interior veneers typically experience in-plane cracking and detachment, but may also be displaced or detached out-of-plane by direct acceleration. Interior partitions loaded out-of-plane and supported on flexible backup support systems can experience cracking and detachment.

Drift analysis is necessary to establish conformance with drift acceptance criteria related to Performance Level. Nonconformance requires limiting structural drift, or special detailing to isolate the veneer support system from the structure to permit drift; this generally requires disassembly of the support system and veneer replacement. Inadequately adhered veneer must be replaced.

11.9.3.3 Acceptance Criteria

Life Safety Performance Level. Compliance is provided by design of the attachment to the backing to meet the out-of-plane force provisions of Section 11.7.3 or 11.7.4, and to meet the in-plane relative displacement drift provisions of Section 11.7.5. The limiting drift ratio is 0.02.

Immediate Occupancy Performance Level.
Compliance is provided by design of the attachment to the backing to meet the out-of-plane force provisions of Section 11.7.3 or 11.7.4, and to meet the in-plane relative displacement drift provisions of Section 11.7.5. The limiting drift ratio is 0.01.

11.9.3.4 Evaluation Requirements

The backup wall or other support and the attachment to that support must be considered, as well as the condition of the veneer itself.

11.9.4 Ceilings

11.9.4.1 Definition and Scope

Ceilings are horizontal and sloping assemblies of materials attached to or suspended from the building structure, or separately supported. Ceilings in an exterior location are referred to as soffits; these provisions also apply to them. Ceilings are mainly of the following types:

Category a. Surface-applied or furred with materials such as wood or metal furring acoustical tile, gypsum board, plaster, or metal panel ceiling materials, which are applied directly to wood joists, concrete slabs, or steel decking with mechanical fasteners or adhesives

Category b. Short dropped gypsum board sections attached to wood or metal furring supported by carrier members

Category c. Suspended metal lath and plaster

Category d. Suspended acoustical board inserted within T-bars, together with lighting fixtures and mechanical items, to form an integrated ceiling system

Some older buildings have heavy decorative ceilings of molded plaster, which may be directly attached to the structure or suspended; these are typically Category a or Category c ceilings.

11.9.4.2 Component Behavior and Rehabilitation Concepts

Ceiling systems are both acceleration- and deformation-sensitive. Surface-applied or furred ceilings are primarily influenced by the performance of their supports. Rehabilitation of the ceiling takes the form of ensuring good attachment and adhesion. Metal lath and plaster ceilings depend on their attachment and bracing for large ceiling areas. Analysis is necessary to establish the acceleration forces and deformations that must be accommodated. Suspended integrated ceilings are highly susceptible to damage, if not braced, with distortion of grid and loss of panels; however, this is not regarded as a life safety threat with lightweight panels (less than two pounds per square foot).

Rehabilitation takes the form of bracing, attachment, and edge details to prescriptive design standards such as the CISCA recommendations appropriate to the seismic zone (CISCA, 1990, 1991).

11.9.4.3 Acceptance Criteria

Life Safety Performance Level. There are no requirements for ceiling Categories a, b, and d, except as noted in the footnotes to Table 11-1. Where rehabilitation is required for ceiling Categories a and b, strengthening to meet force provisions of Section 11.7.3 or 11.7.4 provides compliance. For ceiling Category c, rehabilitation must also comply with relative displacement provisions of Section 11.7.5. Where rehabilitation is required for ceiling Category d, rehabilitation by the Prescriptive Procedure provides compliance.

Immediate Occupancy Performance Level. For ceiling Categories a and b, strengthening to meet force provisions of Section 11.7.3 or 11.7.4 provides compliance. For ceiling Category c, rehabilitation must also comply with relative displacement provisions of Section 11.7.5. For ceiling Category d, rehabilitation by the Prescriptive Procedure provides compliance.

11.9.4.4 Evaluation Requirements

The condition of the ceiling finish material and its attachment to the ceiling support system, the attachment and bracing of the ceiling support system to the structure, and the potential seismic impacts of other nonstructural systems on the ceiling system must be evaluated.

11.9.5 Parapets and Appendages

11.9.5.1 Definition and Scope

Parapets and appendages include exterior nonstructural features that project above or away from a building. They include sculpture and ornament in addition to concrete, masonry, or terra cotta parapets. The following parapets and appendages are within the scope of these requirements:

- Unreinforced masonry parapets more than one and a half times as high as they are thick
- Reinforced masonry parapets more than three times as high as they are wide
- Cornices or ledges constructed of stone, terra cotta, or brick, unless supported by steel or reinforced concrete structure
- Other appendages, such as flagpoles and signs, that are similar to the above in size, weight, or potential consequence of failure

11.9.5.2 Component Behavior and Rehabilitation Concepts

Parapets and appendages are acceleration-sensitive in the out-of-plane direction. Materials or components that are not properly braced may become disengaged and topple; the results are among the most seismically serious consequences of any nonstructural components.

Prescriptive design strategies for masonry parapets not exceeding four feet in height consist of bracing in accordance with the concepts shown in FEMA 74 (FEMA, 1994) and FEMA 172 (BSSC, 1992a), with detailing to conform to accepted engineering practice. Braces for parapets should be spaced at a maximum of eight feet on center, and, when the parapet construction is discontinuous, a continuous backing element should be provided. Where there is no adequate connection, roof construction should be tied to parapet walls at the roof level. Other parapets and appendages should be analyzed for acceleration forces, and braced and connected according to accepted engineering principles.

11.9.5.3 Acceptance Criteria

Life Safety Performance Level. Compliance is provided by strengthening and bracing to a prescriptive concept with engineering evaluation or design to meet the force provisions of Section 11.7.3 or 11.7.4.

Immediate Occupancy Performance Level.
Compliance is similar to that for the Life Safety Performance Level.

11.9.5.4 Evaluation Requirements

Evaluation of masonry parapets should consider the condition of mortar and masonry, connection to supports, type and stability of the supporting structure, and horizontal continuity of the parapet coping.

11.9.6 Canopies and Marquees

11.9.6.1 Definition and Scope

Canopies are projections from an exterior wall to provide weather protection. They may be extensions of the horizontal building structure, or independent structures that are sometimes also tied to the building. Marquees are free-standing structures, often constructed of metal and glass, providing weather protection. Canopies and marquees included within the scope of this document are those that project over exits or exterior walkways, and those with sufficient mass to generate significant seismic forces. Specifically excluded are canvas or other fabric projections.

11.9.6.2 Component Behavior and Rehabilitation Concepts

Canopies and marquees are acceleration-sensitive. Their variety of design is so great that they must be independently analyzed and evaluated for their ability to withstand seismic forces. Rehabilitation may take the form of improving attachment to the building structure, strengthening, bracing, or a combination of measures.

11.9.6.3 Acceptance Criteria

Life Safety Performance Level. Compliance is provided by design to meet the force provisions of Section 11.7.3 or 11.7.4. Consider both horizontal and vertical accelerations.

Immediate Occupancy Performance Level. Compliance is similar to that for the Life Safety Performance Level.

11.9.6.4 Evaluation Requirements

Evaluation should consider buckling in bracing, connection to supports, and type and stability of the supporting structure.

11.9.7 Chimneys and Stacks

11.9.7.1 Definition and Scope

Chimneys and stacks that are cantilevered above building roofs are included within the scope of this document. Light metal residential chimneys, whether enclosed within other structures or not, are not included.

11.9.7.2 Component Behavior and Rehabilitation Concepts

Chimneys and stacks are acceleration-sensitive, and may fail through flexure, shear, or overturning. They may also disengage from adjoining floor or roof structures and damage them, and their collapse or overturning may also damage adjoining structures. Rehabilitation may take the form of strengthening and/or bracing and material repair. Residential chimneys may be braced in accordance with the concepts shown in FEMA 74 (FEMA, 1994).

11.9.7.3 Acceptance Criteria

Life Safety Performance Level. Compliance is provided by strengthening and bracing to a prescriptive concept with engineering evaluation or design to meet the force provisions of Section 11.7.3 or 11.7.4.

Immediate Occupancy Performance Level. Compliance is similar to that for the Life Safety Performance Level.

11.9.7.4 Evaluation Requirements

Evaluation of masonry chimneys should consider the condition of mortar and masonry, connection to adjacent structure, and type and stability of foundations.

Concrete should be evaluated for spalling and exposed reinforcement; steel should be evaluated for corrosion.

11.9.8 Stairs and Stair Enclosures

11.9.8.1 Definition and Scope

Stairs included within the scope of this document are defined as the treads, risers, and landings that make up passageways between floors, as well as the surrounding shafts, doors, windows, and fire-resistant assemblies that constitute the stair enclosure.

11.9.8.2 Component Behavior and Rehabilitation Concepts.

Stairs include a variety of separate components that can be either acceleration- or deformation-sensitive. The stairs themselves may be independent of the structure, or integral with the structure. If integral, they should form part of the overall structural evaluation and analysis, with particular attention paid to the possibility of response modification due to localized stiffness. If independent, the stairs must be evaluated for normal stair loads and their ability to withstand direct acceleration or loads transmitted from the structure through connections.

Stair enclosure materials may fall and render the stairs unusable due to debris.

Rehabilitation of integral or independent stairs may take the form of necessary structural strengthening or bracing, or the introduction of connection details to eliminate or reduce interaction between stairs and the building structure.

Rehabilitation of enclosing walls or glazing should follow the requirements of the relevant sections of this document.

11.9.8.3 Acceptance Criteria

Life Safety Performance Level. Stairs shall meet the force provisions of Section 11.7.3 or 11.7.4 and relative displacement provisions of Section 11.7.5. Other elements of the stair assemblage shall meet the Life Safety acceptance criteria for applicable sections of this chapter.

Immediate Occupancy Performance Level. Stairs shall meet the force provisions of Section 11.7.3 or 11.7.4 and relative displacement provisions of Section 11.7.5. Other elements of the stair assemblage shall meet the applicable Immediate Occupancy acceptance criteria for applicable sections of this chapter.

11.9.8.4 Evaluation Requirements

Evaluation of individual stair elements should consider the materials and condition of stair members and their connections to supports, and the types and stability of supporting and adjacent walls, windows, and other portions of the stair shaft system.

11.10 Mechanical, Electrical, and Plumbing Components: Definition, Behavior, and Acceptance Criteria

11.10.1 Mechanical Equipment

11.10.1.1 Definition and Scope

Equipment that is used for the operation of the building, and is therefore an integral part of it, is included within the scope of the *Guidelines*. Included are:

1. All equipment weighing over 400 pounds
2. Unanchored equipment weighing over 100 pounds that does not have a factor of safety against overturning of 1.5 or greater when design loads, as required by the *Guidelines*, are applied
3. Equipment weighing over 20 pounds that is attached to ceiling, wall, or other support more than four feet above the floor
4. Building operation equipment not included in one of the three categories above

These categories of equipment include, but are not limited to:

- Boilers and furnaces
- Conveyors (nonpersonnel)
- HVAC system equipment, vibration-isolated
- HVAC system equipment, non-vibration-isolated
- HVAC system equipment mounted in-line with ductwork

Equipment such as manufacturing or processing equipment related to the occupant’s business, should be evaluated separately for the effects that failure due to a seismic event could have on the operation of the building.

11.10.1.2 Component Behavior and Rehabilitation Concepts

Mechanical equipment is acceleration-sensitive. Failure of these components consists of sliding, tilting, or overturning of floor- or roof-mounted equipment off its base, and possible loss of attachment (with consequent falling) for equipment attached to a vertical structure or suspended, and failure of piping or electrical wiring connected to the equipment.

Construction of mechanical equipment to nationally recognized codes and standards, such as those approved by the American National Standards Institute, provides adequate strength to accommodate all normal and upset operating loads.

Basic rehabilitation consists of securely anchoring floor-mounted equipment by bolting, with detailing appropriate to the base construction of the equipment.

Seismic forces can be established by analysis using the default Equation 11-1. Equipment weighing over 400 pounds and located on the third floor or above (or on an equivalent-height roof) should be analyzed using Equations 11-2 and 11-3.

Existing attachments for attached or suspended equipment must be evaluated for seismic load capacity, and strengthened or braced as necessary. Attachments that provide secure anchoring eliminate or reduce the likelihood of piping or electrical distribution failure.

11.10.1.3 Acceptance Criteria

Life Safety Performance Level. Equipment anchorage should meet the force provisions of Section 11.7.3 or 11.7.4.

Immediate Occupancy Performance Level.
Compliance criteria are similar to those for the Life Safety Performance Level.

11.10.1.4 Evaluation Requirements

Equipment must be analyzed to establish acceleration-induced forces, and visually evaluated for the existence of satisfactory supports, hold-downs, and bracing. Existing concrete anchors may have to be tested by applying torque to the nuts to confirm that adequate strength is present.

11.10.2 Storage Vessels and Water Heaters

11.10.2.1 Definition and Scope

This section includes all vessels that contain fluids used for building operation. The vessel may be fabricated of materials such as steel or other metals, or fiberglass, or it may be a glass-lined tank. These requirements may also be applied, with judgment, to vessels that contain solids that act as a fluid, and vessels containing fluids not involved in the operation of the building.

Vessels are classified into two categories:

Category 1. Vessels with structural support of contents, in which the shell is supported by legs or a skirt

Category 2. Flat bottom vessels in which the weight of the contents is supported by the floor, roof, or a structural platform

11.10.2.2 Component Behavior and Rehabilitation Concepts

Tanks and vessels are acceleration-sensitive. Category 1 vessels fail by stretching of anchor bolts, buckling and disconnection of supports, and consequent tilting or overturning of the vessel. A Category 2 vessel may be displaced from its foundation, or its shell may fail by yielding near the bottom, creating a visible bulge, or possible leakage. Displacement of both types of vessel may cause rupturing of connecting piping and leakage.

Category 1 residential water heaters with a capacity no greater than 100 gallons may be rehabilitated by prescriptive design methods, such as concepts shown in FEMA 74 (FEMA, 1994) or FEMA 172 (BSSC, 1992a). Category 1 vessels with a capacity less than 1000 gallons should be designed to meet the force provisions of Section 11.7.3 or 11.7.4, and bracing strengthened or added as necessary. Other Category 1 and Category 2 vessels should be evaluated against a recognized standard, such as API STD 650-93 or API-90 by the American Petroleum Institute (API, 1993), for vessels containing petroleum products or other chemicals, or AINSI/AWWA D100-96 (AWS D5 2-96) by the American Water Works Association (AWWA, 1996), for water vessels.

11.10.2.3 Acceptance Criteria

Life Safety Performance Level.
Category 1 equipment. Refer to Table 11-1 for applicability. Design and support to meet the force provisions of Section 11.7.3 or 11.7.4 will provide compliance.

Category 2 equipment. Design in accordance with a recognized prescriptive standard and to meet force provisions of Section 11.7.3 or 11.7.4 provides compliance.

Immediate Occupancy Performance Level.
Compliance criteria are similar to those for Life Safety.

11.10.2.4 Evaluation Requirements

All equipment must be visually evaluated to determine the existence of the necessary hold-downs, supports, and bracing. Existing concrete anchors may have to be tested by applying torque to the nuts to confirm that adequate strength is present.

11.10.3 Pressure Piping

11.10.3.1 Definition and Scope

This section includes all piping that carries fluids which, in their vapor stage, exhibit a pressure of 15 psi, gauge, or higher, except fire suppression piping.

11.10.3.2 Component Behavior and Rehabilitation Concepts

Piping is predominantly acceleration-sensitive, but piping that runs between floors or seismic joints may be deformation-sensitive. The most common failure is joint failure, caused by inadequate support or bracing.

Rehabilitation is accomplished by prescriptive design approaches to support and bracing. The prescriptive requirements of *NFPA-13* (NFPA, 1996) should be used. Piping systems should be evaluated for compliance with the latest edition of ASME/ANSI B31.9 and other B31 standards where applicable. For large critical piping systems, the building official or responsible engineer must establish forces and evaluate supports.

11.10.3.3 Acceptance Criteria

Life Safety Performance Level. Design in accordance with a recognized prescriptive standard, and to meet force provisions of Section 11.7.3 or 11.7.4 and displacement provisions of Section 11.7.5, will provide compliance.

Immediate Occupancy Performance Level. Compliance criteria are similar to those for Life Safety.

11.10.3.4 Evaluation Requirements

High-pressure piping shall be tested in accordance with the ASME/ANSI standards mentioned above. In addition to other tests, lines shall be hydrostatically tested to 150% of the maximum anticipated pressure of the system.

11.10.4 Fire Suppression Piping

11.10.4.1 Definition and Scope

Fire suppression piping includes fire sprinkler piping consisting of main risers and laterals weighing, loaded, in the range of 30 to 100 pounds per lineal foot, with branches of decreasing size down to approximately two pounds per foot.

11.10.4.2 Component Behavior and Rehabilitation Concepts

Piping is predominantly acceleration-sensitive, but piping that runs between floors or seismic joints may be deformation-sensitive. The most common failure is joint failure, caused by inadequate support or bracing, or by sprinkler heads impacting adjoining materials.

Rehabilitation is accomplished by prescriptive design approaches to support and bracing. The prescriptive requirements of *NFPA-13* (NFPA, 1996) should be used.

11.10.4.3 Acceptance Criteria

Life Safety Performance Level. Design in accordance with a recognized prescriptive standard to meet force provisions of Section 11.7.3 or 11.7.4. provides compliance.

Immediate Occupancy Performance Level. Compliance criteria are similar to those for Life Safety.

11.10.4.4 Evaluation Requirements

Fire suppression piping must be evaluated for adequate support, flexibility, protection at seismic movement joints, and freedom from impact from adjoining materials at the sprinkler heads. The support and bracing of bends of the main risers and laterals, as well as maintenance of adequate flexibility to prevent buckling, are especially important.

11.10.5 Fluid Piping other than Fire Suppression

11.10.5.1 Definition and Scope

This section includes all piping, other than pressure piping or fire suppression lines, that transfers fluids under pressure by gravity, or is open to the atmosphere. This includes drainage and ventilation piping; hot, cold, and chilled water piping; and piping carrying liquids, as well as fuel gas lines, used in industrial, medical, laboratory, and other occupancies. There are two categories of fluids, based on potential damage or hazard to personnel:

Category 1. Hazardous materials and flammable liquids that would pose an immediate life safety danger if exposed, because of inherent properties of the contained material, as defined in NFPA 325-94, 49-94, 491M-91, and 704-90.

Category 2. Materials that, in case of line rupture, would cause property damage, but pose no immediate life safety danger.

11.10.5.2 Component Behavior and Rehabilitation Concepts

Piping is predominantly acceleration-sensitive, but piping that runs between floors or expansion or seismic joints may be deformation-sensitive. The most common failure is joint failure, caused by inadequate support or bracing.

Category 1 piping rehabilitation is accomplished by strengthening support and bracing, using the prescriptive methods of SP-58 (MSS, 1993); the piping systems themselves should be designed to meet the force provisions of Sections 11.7.3 or 11.7.4 and relative displacement provisions of Section 11.7.5. The effects of temperature differences, dynamic fluid forces, and piping contents should be taken into account.

Category 2 piping rehabilitation is accomplished by strengthening support and bracing, using the prescriptive methods of SP-58 (MSS, 1993) as long as the piping falls within the size limitations of those guidelines. Piping that exceeds the limitations of those guidelines shall be designed to meet the force provisions of Section 11.7.3 or 11.7.4 and relative displacement provisions of Section 11.7.5.

11.10.5.3 Acceptance Criteria

Life Safety Performance Level
Category 1 piping systems. Design to meet prescriptive standards, the force provisions of Section 11.7.3 or 11.7.4, and the relative displacement provisions of Section 11.7.5, provides compliance.

Category 2 piping systems. Design to meet prescriptive standards provides compliance.

Immediate Occupancy Performance Level. Acceptance criteria are similar to those for Life Safety. Prescriptive standards should be met for essential facilities.

11.10.5.4 Evaluation Requirements

Piping must be evaluated for adequate support, flexibility, and protection at seismic movement joints. The support and bracing of bends in the main risers and laterals, as well as maintenance of adequate flexibility to prevent buckling, are especially important. Piping must be protected by adequate insulation from detrimental heat effects.

11.10.6 Ductwork

11.10.6.1 Definition and Scope

This section includes HVAC and special exhaust ductwork systems. Seismic restraints are not required for ductwork that is not conveying hazardous materials, and that meets either of the following conditions.

- HVAC ducts are suspended from hangers 12 inches or less in length from the top of the duct to the supporting structure. The hangers should be designed and placed in such a way as to avoid significant bending of the hangers.
- HVAC ducts have a cross-sectional area of less than six square feet.

11.10.6.2 Component Behavior and Rehabilitation Concepts

Ducts are predominantly acceleration-sensitive, but when ductwork runs between floors or across expansion or seismic joints it may be deformation-sensitive.

Damage is caused by failure of supports or lack of bracing that causes deformation or rupture of the ducts at joints, leading to leakage from the system.

Rehabilitation consists of strengthening supports and strengthening or adding bracing. Prescriptive design methods may be used, per SMACNA Duct Construction Standards (SMACNA, 1980, 1985).

11.10.6.3 Acceptance Criteria

Life Safety Performance Level. Design to meet prescriptive standards provides compliance.

Immediate Occupancy Performance Level.
Compliance criteria are similar to those for Life Safety. Prescriptive standards should be for essential facilities.

11.10.6.4 Evaluation Requirements

These components must be evaluated by visual means to ascertain their compliance with the conditions defined in Section 11.10.6.1.

11.10.7 Electrical and Communications Equipment

11.10.7.1 Definition and Scope

This section includes all electrical and communication equipment, including panel boards, battery racks, motor control centers, switch gear, and other fixed components located in electrical rooms or elsewhere in the building.

The following equipment is subject to these *Guidelines*:

1. All equipment weighing over 400 pounds

2. Unanchored equipment weighing over 100 pounds that does not have a factor of safety against overturning of 1.5 or greater when design loads, as required by the *Guidelines*, are applied

3. Equipment weighing over 20 pounds that is attached to ceiling, wall, or other support more than four feet above the floor

4. Building operation equipment not falling into one of the three categories above

11.10.7.2 Component Behavior and Rehabilitation Concepts

Electrical equipment is acceleration-sensitive. Failure of these components consists of sliding, tilting, or overturning of floor- or roof-mounted equipment off its base, and possible loss of attachment (with consequent falling) for equipment attached to a vertical structure or suspended, and failure of electrical wiring connected to the equipment.

Construction of electrical equipment to nationally recognized codes and standards, such as those approved by ANSI, provides adequate strength to accommodate all normal and upset operating loads.

Basic rehabilitation consists of securely anchoring floor-mounted equipment by bolting, with detailing appropriate to the base construction of the equipment.

11.10.7.3 Acceptance Criteria

Life Safety Performance Level. Design to meet the force provisions of Section 11.7.3 or 11.7.4 provides compliance.

Immediate Occupancy Performance Level. Acceptance criteria are similar to those for Life Safety.

11.10.7.4 Evaluation Requirements

Equipment should be visually evaluated to determine its category, and the existence of the necessary hold-downs, supports, and braces. Larger equipment requiring the Analytical Procedure must be analyzed to determine forces, and visually evaluated. Concrete anchors may have to be tested by applying torque to the nuts to confirm that adequate strength is present.

11.10.8 Electrical and Communications Distribution Components

11.10.8.1 Definition and Scope

This includes all electrical and communications transmission lines, conduit, and cables, and their supports.

11.10.8.2 Component Behavior and Rehabilitation Concepts

Electrical distribution equipment is predominantly acceleration-sensitive, but wiring or conduit that runs between floors or expansion or seismic joints may be deformation-sensitive. Failure occurs most commonly by inadequate support or bracing, deformation of the attached structure, or impact from adjoining materials.

Rehabilitation is accomplished by strengthening support and bracing using the prescriptive methods contained in SMACNA standards (SMACNA, 1980, 1985).

11.10.8.3 Acceptance Criteria

Life Safety Performance Level. Design to meet prescriptive standards provides compliance.

Immediate Occupancy Performance Level. Acceptance criteria are similar to those for Life Safety. Prescriptive standards should be for essential facilities.

11.10.8.4 Evaluation Requirements

Components should be visually evaluated to determine the existence of necessary supports and bracing.

11.10.9 Light Fixtures

11.10.9.1 Definition and Scope

This section includes lighting fixtures in the following categories:

Category 1. Recessed in ceilings

Category 2. Surface mounted to ceilings or walls

Category 3. Supported within a suspended ceiling system (integrated ceiling)

Category 4. Suspended from ceilings or structure (pendant or chain)

11.10.9.2 Component Behavior and Rehabilitation Concepts

Failure of Category 1 and 2 components occurs through failure of attachment of the light fixture and/or failure of the supporting ceiling or wall. Failure of Category 3 components occurs through loss of support from the T-bar system, and by distortion caused by deformation of the supporting structure or deformation of the ceiling grid system, allowing the fixture to fall. Failure of Category 4 components is caused by excessive swinging that results in the pendant or chain support breaking on impact with adjacent materials, or the support being pulled out of the ceiling.

Rehabilitation of Category 1 and 2 components involves attachment repair or fixture replacement in association with necessary rehabilitation of the supporting ceiling or wall. Rehabilitation of Category 3 components involves the addition of independent support for the fixture from the structure or substructure in accordance with FEMA 74 design concepts (FEMA, 1994). Rehabilitation of Category 4 components involves strengthening of attachment and ensuring freedom to swing without impacting adjoining materials.

11.10.9.3 Acceptance Criteria

Life Safety Performance Level
Categories 1 and 2. There are no specific acceptance criteria, but secure connection to ceiling or wall must be assured.

Category 3. Systems bracing and support to meet prescriptive requirements provides compliance.

Category 4. Fixtures weighing over 20 pounds should have adequate articulating or ductile connections to the building, and be free to swing without impacting adjoining materials.

Immediate Occupancy Performance Level. Acceptance criteria are similar to those for Life Safety. Prescriptive standards should be met for essential facilities.

11.10.9.4 Evaluation Requirements

Fixtures must be visually evaluated to determine the adequacy of supports and, for Category 3 fixtures, the existence of adequate independent support.

11.11 Furnishings and Interior Equipment: Definition, Behavior, and Acceptance Criteria

11.11.1 Storage Racks

11.11.1.1 Definition and Scope

Storage racks include systems, usually constructed of metal, for the purpose of holding materials either permanently or temporarily. Storage racks are generally purchased as proprietary systems installed by a tenant and are often not under the direct control of the building owner. Thus, they are usually not part of the construction contract, and often have no foundation or foundation attachment. However, they are often permanently installed, and their size and loaded weight make them an important hazard to either life, property, or the surrounding structure. Storage racks in excess of four feet in height located in occupied locations shall be considered when the Life Safety Performance Level is selected.

11.11.1.2 Component Behavior and Rehabilitation Concepts

Storage racks are acceleration-sensitive, and may fail internally—through inadequate bracing or moment-resisting capacity—or externally, by overturning caused by absence or failure of foundation attachments.

Rehabilitation is usually accomplished by the addition of bracing to the rear and side panels of racks and/or by improving the connection of the rack columns to the supporting slab. In rare instances, foundation improvements may be required to remedy insufficient bearing or uplift load capacity.

Seismic forces can be established by analysis in accordance with Section 11.7.3 or 11.7.4. However, special attention should be paid to the evaluation and analysis of large, heavily loaded rack systems because of their heavy loading and lightweight structural members.

11.11.1.3 Acceptance Criteria

Life Safety Performance Level. Design to meet the force provisions of Section 11.7.3 or 11.7.4 provides compliance.

Immediate Occupancy Performance Level. Acceptance criteria are similar to those for Life Safety.

11.11.1.4 Evaluation Requirements

Evaluation should consider buckling or racking failure of rack elements, connection to support structures, and type and stability of supporting structure.

11.11.2 Bookcases

11.11.2.1 Definition and Scope

Bookcases, constructed of wood or metal, in excess of four feet high should be considered.

11.11.2.2 Component Behavior and Rehabilitation Concepts

Bookcases are acceleration-sensitive, and may deform or overturn due to inadequate bracing or attachment to floors or adjacent walls, columns, or other structural members. Rehabilitation is usually accomplished by the addition of metal cross bracing to the rear of the bookcase to improve its internal resistance to racking forces, and by bracing the bookcase both in- and out-of-plane to the adjacent structure or walls to prevent overturning and racking.

11.11.2.3 Acceptance Criteria

Life Safety Performance Level. Design to meet the force provisions of Section 11.7.3 or 11.7.4 provides compliance.

Immediate Occupancy Performance Level. Acceptance criteria are similar to those for Life Safety.

11.11.2.4 Evaluation Requirements

Evaluation should consider the loading, type, and condition of bookcases, their connection to support structures, and type and stability of supporting structure.

11.11.3 Computer Access Floors

11.11.3.1 Definition and Scope

Computer access floors are panelized, elevated floor systems designed to facilitate access to wiring, fiber optics, and other services associated with computers and other electronic components. Access floors vary in height but generally are less than three feet above the supporting structural floor. The systems include structural legs, horizontal panel supports, and panels.

11.11.3.2 Component Behavior and Rehabilitation Concepts

These components are both acceleration- and deformation-sensitive. They may displace laterally or buckle vertically under seismic loads. Rehabilitation of access floors usually includes a combination of improved attachment of computer and communication racks through the access floor panels to the supporting steel structure or to the underlying floor system, while improving the lateral-load-carrying capacity of the steel stanchion system by installing braces or improving the connection of the stanchion base to the supporting floor, or both.

Rehabilitation should be designed in accordance with concepts described in FEMA 74 (FEMA, 1994). The weight of the floor system, as well as supported equipment, should be included in the analysis.

11.11.3.3 Acceptance Criteria

Life Safety Performance Level. Not applicable.

Immediate Occupancy Performance Level. Design to meet force provisions of Section 11.7.3 or 11.7.4 provides compliance, together with design to approved prescriptive standards.

11.11.3.4 Evaluation Requirements

Evaluation should consider buckling and racking of access floor supports, and connection to the support structure. The effects of mounted equipment, including possible future equipment, should also be considered.

11.11.4 Hazardous Materials Storage

11.11.4.1 Definition and Scope

For the purpose of this section, hazardous materials storage shall be defined as permanently installed containers—either freestanding, on supports, or stored on countertops or shelves—that hold materials defined to be hazardous by the National Institute for Occupational Safety and Health, including the following types:

- Propane gas tanks
- Compressed gas vessels
- Dry or liquid chemical storage containers

Large nonbuilding structures, such as large tanks found in heavy industry or power plants, floating-roof oil storage tanks, and large (greater than ten feet long) propane tanks at propane manufacture or distribution plants are not within the scope of these *Guidelines.*

11.11.4.2 Component Behavior and Rehabilitation Concepts

These components are acceleration-sensitive; upset of the storage container may release the hazardous material. Failure occurs because of buckling and overturning of supports and/or inadequate bracing. Rehabilitation consists of strengthening and increasing or adding bracing designed according to concepts described in FEMA 74 (FEMA, 1994) and FEMA 172 (BSSC, 1992a).

11.11.4.3 Acceptance Criteria

Life Safety Performance Level. Design to approved prescriptive concepts provides compliance.

Immediate Occupancy Performance Level. Acceptance criteria are similar to those for the Life Safety Performance Level. Prescriptive standards should be met for essential facilities.

11.11.4.4 Evaluation Requirements

Evaluation should consider the location and types of hazardous materials, container materials, manner of bracing, internal lateral resistance, and the effect of hazardous material spills.

11.11.5 Computer and Communication Racks

11.11.5.1 Definition and Scope

Computer and communication racks are large, free-standing rack systems designed to support computer and other electronic equipment. Racks may be supported on either structural or access floors and may or may not be attached directly to these supports. The equipment itself is not included in this definition. All computer and communication racks are included within the scope of this section.

11.11.5.2 Component Behavior and Rehabilitation Concepts

These components are acceleration-sensitive, and may fail internally—through inadequate bracing or moment-resisting capacity—or externally, by overturning caused by absence or failure of floor attachments.

Rehabilitation is usually accomplished by the addition of bracing to the rear and side panels of the racks, and/or by improving the connection of the rack to the supporting floor using concepts shown in FEMA 74 (FEMA, 1994) or FEMA 172 (BSSC, 1992a).

11.11.5.3 Acceptance Criteria

Life Safety Performance Level. Not applicable.

Immediate Occupancy Performance Level. Design to meet force provisions of Section 11.7.3 or 11.7.4 provides compliance, together with design to approved prescriptive standards.

11.11.5.4 Evaluation Requirements

Evaluation should consider buckling or racking failure of rack elements, their connection to support structures, and type and stability of the supporting structure. The effect of rack failure on equipment should also be considered.

11.11.6 Elevators

11.11.6.1 Definition and Scope

Elevators include cabs and shafts, as well as all equipment and equipment rooms associated with elevator operation, such as hoists, counterweights, cables, and controllers.

11.11.6.2 Component Behavior and Rehabilitation Concepts

Most elements of elevators are acceleration-sensitive, and can become dislodged or derailed. Shafts and hoistway rails, which rise through a number of floors, may also be deformation-sensitive. Shaft walls and the construction of machinery room walls are often not engineered and must be considered in a way similar to that for other partitions. Shaft walls that are of unreinforced masonry or hollow tile must be considered with special care, since failure of these elements violates Life Safety Performance Level criteria.

Elevator machinery may be subject to the same damage as other heavy floor-mounted equipment. Electrical power loss renders elevators inoperable.

Rehabilitation measures include a variety of techniques taken from specific component sections for partitions, controllers, and machinery. Rehabilitation specific to elevator operation can include seismic shutoffs, cable restrainers, and counterweight retainers; such measures should be in accordance with ASME A17.1 (ASME, 1996).

11.11.6.3 Acceptance Criteria

Life Safety Performance Level. Design to meet force provisions of Section 11.7.3 or 11.7.4 provides compliance, together with design to approved prescriptive standards.

Immediate Occupancy Performance Level. Rehabilitation criteria are similar to those for Life Safety.

11.11.6.4 Evaluation Requirements

Evaluation should consider the construction of elevator shafts consistent with the requirements of applicable sections of the *Guidelines*. The possibility of displacement or derailment of hoistway counterweights and cables should be considered, as should the anchorage of elevator machinery.

11.11.7 Conveyors

11.11.7.1 Definition and Scope

Conveyors are defined as material conveyors only for the purposes of this section, including all machinery and controllers necessary to operation.

11.11.7.2 Component Behavior and Rehabilitation Concepts

Conveyors are both acceleration- and deformation-sensitive. Conveyor machinery may be subject to the same damage as other heavy floor-mounted equipment. In addition, deformation of adjoining building materials may render the conveyor inoperable. Electrical power loss renders the conveyor inoperable.

Rehabilitation of the conveyor involves Prescriptive Procedures using special skills provided by the conveyor manufacturer.

11.11.7.3 Acceptance Criteria

Life Safety Performance Level. Not applicable.

Immediate Occupancy Performance Level. Design to meet force provisions of Section 11.7.3 or 11.7.4 and displacement provisions of Section 11.7.5, together with special prescriptive concepts, provides compliance.

11.11.7.4 Evaluation Requirements

Evaluation should consider the stability of machinery consistent with the requirements of applicable sections of these *Guidelines*.

11.12 Definitions

Acceleration-sensitive nonstructural component: A nonstructural component sensitive to and subject to damage from inertial loading. Once inertial loads are generated within the component, the deformation of the component may be significant; this is separate from the issue of deformation imposed on the component by structural deflections (see deformation-sensitive nonstructural components).

Component, flexible: A component, including its attachments, having a fundamental period greater than 0.06 seconds.

Component, rigid: A component, including its attachments, having a fundamental period less than or equal to 0.06 seconds.

Contents: Movable items within the building introduced by the owner or occupants.

Deformation-sensitive nonstructural component: A nonstructural component sensitive to deformation imposed on it by the drift or deformation of the structure, including deflection or deformation of diaphragms.

Flexible connections: Connections between components that permit rotational and/or translational movement without degradation of performance. Examples include universal joints, bellows expansion joints, and flexible metal hose.

Nonstructural component: An architectural, mechanical, plumbing, or electrical component, or item of interior equipment and furnishing, permanently installed in the building, as listed in Table 11-1.

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

11.13 Symbols

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

D_r Drift ratio

F_p Seismic design force applied horizontally at the component's center of gravity and distributed according to the component's mass distribution

R_p Component response modification factor, related to ductility of anchorage that varies from 1.25 to 6.0 (select appropriate value from Table 11-2)

S_{XS} Spectral response acceleration at short periods for any hazard level and any damping, g

W_p Component operating weight

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

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

a_p Component amplification factor, related to rigidity of component, that varies from 1.00 to 2.50 (select appropriate value from Table 11-2)

h Average roof elevation of structure, relative to grade elevation

x Elevation in structure of component relative to grade elevation

δ_{xA} Deflection at building level x of Building A, determined by an elastic analysis as defined in Chapter 3

δ_{yA} Deflection at building level y of Building A, determined by an elastic analysis as defined in Chapter 3

δ_{xB} Deflection at building level x of Building B, determined by an elastic analysis as defined in Chapter 3

11.14 References

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A. Glossary

A

Acceleration-sensitive nonstructural component: A nonstructural component sensitive to and subject to damage from inertial loading. Once inertial loads are generated within the component, the deformation of the component may be significant; this is separate from the issue of deformation imposed on the component by structural deflections (see deformation-sensitive nonstructural components**).**

Acceptance criteria: Permissible values of such properties as drift, component strength demand, and inelastic deformation, used to determine the acceptability of a component's projected behavior at a given Performance Level.

Action: Sometimes called a generalized force, most commonly a single force or moment. However, an action may also be a combination of forces and moments, a distributed loading, or any combination of forces and moments. Actions always produce or cause displacements or deformations; for example, a bending moment action causes flexural deformation in a beam; an axial force action in a column causes axial deformation in the column; a torsional moment action on a building causes torsional deformations (displacements) in the building.

Allowable bearing capacity: Foundation load or stress commonly used in working-stress design (often controlled by long-term settlement rather than soil strength).

Aspect ratio: Ratio of height to width for vertical diaphragms, and width to depth for horizontal diaphragms.

Assembly: A collection of structural members and/or components connected in a such manner that load applied to any one component will affect the stress conditions of adjacent components.

B

Balloon framing: Continuous stud framing from sill to roof, with intervening floor joists nailed to studs and supported by a let-in ribbon. (See platform framing.)

Base: The level at which earthquake effects are considered to be imparted to the building.

Beam: A structural member whose primary function is to carry loads transverse to its longitudinal axis, usually a horizontal member in a seismic frame system.

Bearing wall: A wall that supports gravity loads of at least 200 pounds per linear foot from floors and/or roofs.

Bed joint: The horizontal layer of mortar on which a masonry unit is laid.

Boundary component (boundary member): A member at the perimeter (edge or opening) of a shear wall or horizontal diaphragm that provides tensile and/or compressive strength.

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

Braced frame: An essentially vertical truss system of concentric or eccentric type that resists lateral forces.

BSE-1: Basic Safety Earthquake-1, which is the lesser of the ground shaking at a site for a 10%/50 year earthquake or two-thirds of the Maximum Considered Earthquake (MCE) at the site.

BSE-2: Basic Safety Earthquake-2, which is the ground shaking at a site for an MCE.

BSO: Basic Safety Objective, a Rehabilitation Objective in which the Life Safety Performance Level is reached for the BSE-1 demand and the Collpase Prevention Performance Level is reached for the BSE-2.

Building Performance Level: A limiting damage state, considering structural and nonstructural building components, used in the definition of Rehabilitation Objectives.

C

Capacity: The permissible strength or deformation for a component action.

Cavity wall: A masonry wall with an air space between wythes. Wythes are usually joined by wire reinforcement, or steel ties. Also known as a noncomposite wall.

Chevron bracing: See V-braced frame.

Chord: See diaphragm chord.

Clay tile masonry: Masonry constructed with hollow units made of clay tile. Typically, units are laid with cells running horizontally, and are thus ungrouted. In some cases, units are placed with cells running vertically, and may or may not be grouted.

Clay-unit masonry: Masonry constructed with solid, cored, or hollow units made of clay. Hollow clay units may be ungrouted, or grouted.

Coefficient of variation: For a sample of data, the ratio of the standard deviation for the sample to the men value for the sample.

Collar joint: Vertical longitudinal joint between wythes of masonry or between masonry wythe and back-up construction that may be filled with mortar or grout.

Collector: See drag strut.

Column (or beam) jacketing: A method in which a concrete column or beam is covered with a steel or concrete "jacket" in order to strengthen and/or repair the member by confining the concrete.

Components: The basic structural members that constitute the building, such as beams, columns, slabs, braces, piers, coupling beams, and connections. Components, such as columns and beams, are combined to form elements (e.g., a frame).

Component, flexible: A component, including its attachments, having a fundamental period greater than 0.06 seconds.

Component, rigid: A component, including its attachments, having a fundamental period less than or equal to 0.06 seconds.

Composite masonry wall: Multiwythe masonry wall acting with composite action.

Composite panel: A structural panel comprising thin wood strands or wafers bonded together with exterior adhesive.

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

Concrete masonry: Masonry constructed with solid or hollow units made of concrete. Hollow concrete units may be ungrouted, or grouted.

Condition of service: The environment to which the structure will be subjected. Moisture conditions are the most significant issue; however, temperature can have a significant effect on some assemblies.

Connection: A link between components or elements that transmits actions from one component or element to another component or element. Categorized by type of action (moment, shear, or axial), connection links are frequently nonductile.

Contents: Movable items within the building introduced by the owner or occupants.

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

Control node: The node in the mathematical model of a building used to characterize mass and earthquake displacement.

Corrective measure: Any modification of a component or element, or the structure as a whole, intended to reduce building vulnerability.

Coupling beam: Flexural member that ties or couples adjacent shear walls acting in the same plane. A coupling beam is designed to yield and dissipate inelastic energy, and, when properly detailed and proportioned, has a significant effect on the overall stiffness of the coupled wall.

Cripple studs: Short studs between header and top plate at opening in wall framing or studs between base sill and sill of opening.

Cripple wall: Short wall between foundation and first floor framing.

Critical action: That component action that reaches its elastic limit at the lowest level of lateral deflection, or loading, for the structure.

Crosstie: A beam or girder that spans across the width of the diaphragm, accumulates the wall loads, and transfers them, over the full depth of the diaphragms, into the next bay and onto the nearest shear wall or frame.

D

Decay: Decomposition of wood caused by action of wood-destroying fungi. The term "dry rot" is used interchangeably with decay.

Decking: Solid sawn lumber or glued laminated decking, nominally two to four inches thick and four inches and wider. Decking may be tongue-and-groove or connected at longitudinal joints with nails or metal clips.

Deep foundation: Piles or piers.

Deformation: Relative displacement or rotation of the ends of a component or element.

Deformation-sensitive nonstructural component: A nonstructural component sensitive to deformation imposed on it by the drift or deformation of the structure, including deflection or deformation of diaphragms.

Demand: The amount of force or deformation imposed on an element or component.

Design displacement: The design earthquake displacement of an isolation or energy dissipation system, or elements thereof, excluding additional displacement due to actual and accidental torsion.

Design resistance: Resistance (force or moment as appropriate) provided by member or connection; the product of adjusted resistance, the resistance factor, confidence factor, and time effect factor.

Diagonal bracing: Inclined structural members carrying primarily axial load, employed to enable a structural frame to act as a truss to resist horizontal loads.

Diaphragm: A horizontal (or nearly horizontal) structural element used to distribute inertial lateral forces to vertical elements of the lateral-force-resisting system.

Diaphragm chord: A diaphragm component provided to resist tension or compression at the edges of the diaphragm.

Diaphragm collector: A diaphragm component provided to transfer lateral force from the diaphragm to vertical elements of the lateral-force-resisting system or to other portions of the diaphragm.

Diaphragm ratio: See aspect ratio.

Differential compaction: An earthquake-induced process in which loose or soft soils become more compact and settle in a nonuniform manner across a site.

Dimensioned lumber: Lumber from nominal two through four inches thick and nominal two or more inches wide.

Displacement: The total movement, typically horizontal, of a component or element or node.

Displacement restraint system: Collection of structural components and elements that limit lateral displacement of seismically-isolated buildings during the BSE-2.

Displacement-dependent energy dissipation devices: Devices having mechanical properties such that the force in the device is related to the relative displacement in the device.

Dowel bearing strength: The maximum compression strength of wood or wood-based products when subjected to bearing by a steel dowel or bolt of specific diameter.

Dowel type fasteners: Includes bolts, lag screws, wood screws, nails, and spikes.

Drag strut: A component parallel to the applied load that collects and transfers diaphragm shear forces to the vertical lateral-force-resisting components or elements, or distributes forces within a diaphragm. Also called collector, diaphragm strut, or tie.

Dressed size: The dimensions of lumber after surfacing with a planing machine. Usually 1/2 to 3/4 inch less than nominal size.

Dry service: Structures wherein the maximum equilibrium moisture content does not exceed 19%.

Dual system: A structural system included in buildings with the following features:

- An essentially complete space frame provides support for gravity loads.
- Resistance to lateral load is provided by concrete or steel shear walls, steel eccentrically braced frames (EBF), or concentrically braced frames (CBF) along with moment-resisting frames (Special Moment Frames, or Ordinary Moment Frames) that are capable of resisting at least 25% of the lateral loads.
- Each system is also designed to resist the total lateral load in proportion to its relative rigidity.

E

Eccentric braced frame (EBF): A diagonal braced frame in which at least one end of each diagonal bracing member connects to a beam a short distance from a beam-to-column connection or another brace end.

Edge distance: The distance from the edge of the member to the center of the nearest fastener. When a member is loaded perpendicular to the grain, the loaded edge shall be defined as the edge in the direction toward which the fastener is acting.

Effective damping: The value of equivalent viscous damping corresponding to the energy dissipated by the building, or element thereof, during a cycle of response.

Effective stiffness: The value of the lateral force in the building, or an element thereof, divided by the corresponding lateral displacement.

Element: An assembly of structural components that act together in resisting lateral forces, such as moment-resisting frames, braced frames, shear walls, and diaphragms.

Energy dissipation device (EDD): Non-gravity-load-supporting element designed to dissipate energy in a stable manner during repeated cycles of earthquake demand.

Energy dissipation system (EDS): Complete collection of all energy dissipation devices, their supporting framing, and connections.

F

Fault: Plane or zone along which earth materials on opposite sides have moved differentially in response to tectonic forces.

Flexible connections: Connections between components that permit rotational and/or translational movement without degradation of performance. Examples include universal joints, bellows expansion joints, and flexible metal hose.

Flexible diaphragm: A diaphragm that meets requirements of Section 3.2.4.

Footing: A structural component transferring the weight of a building to the foundation soils and resisting lateral loads.

Foundation soils: Soils supporting the foundation system and resisting vertical and lateral loads.

Foundation springs: Method of modeling to incorporate load-deformation characteristics of foundation soils.

Foundation system: Structural components (footings, piles).

Framing type: Type of seismic resisting system.

Fundamental period: The first mode period of the building in the direction under consideration.

G

Gauge or row spacing: The center-to-center distance between fastener rows or gauge lines.

Glulam beam: Shortened term for glued-laminated beam.

Grade: The classification of lumber in regard to strength and utility, in accordance with the grading rules of an approved agency.

Grading rules: Systematic and standardized criteria for rating the quality of wood products.

Gypsum wallboard or drywall: An interior wall surface sheathing material sometimes considered for resisting lateral forces.

H

Hazard level: Earthquake shaking demands of specified severity, determined on either a probabilistic or deterministic basis.

Head joint: Vertical mortar joint placed between masonry units in the same wythe.

Hold-down: Hardware used to anchor the vertical chord forces to the foundation or framing of the structure in order to resist overturning of the wall.

Hollow masonry unit: A masonry unit whose net cross-sectional area in every plane parallel to the bearing surface is less than 75% of the gross cross-sectional area in the same plane.

I

Infill: A panel of masonry placed within a steel or concrete frame. Panels separated from the surrounding frame by a gap are termed "isolated infills". Panels that are in tight contact with a frame around its full perimeter are termed “shear infills.”

In-plane wall: See shear wall.

Inter-story drift: The relative horizontal displacement of two adjacent floors in a building. Inter-story drift can also be expressed as a percentage of the story height separating the two adjacent floors.

Isolation interface: The boundary between the upper portion of the structure (superstructure), 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.

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

J

Joint: Area where two or more ends, surfaces, or edges are attached. Categorized by type of fastener or weld used and method of force transfer.

K

King stud: Full height stud or studs adjacent to openings that provide out-of-plane stability to cripple studs at openings.

L

Landslide: A down-slope mass movement of earth resulting from any cause.

Lateral support member: Member designed to inhibit lateral buckling or lateral-torsional buckling of a component.

Lateral-force-resisting system: Those elements of the structure that provide its basic lateral strength and stiffness, and without which the structure would be laterally unstable.

Light framing: Repetitive framing with small uniformly spaced members.

Linear procedure: Analysis based on a straight-line (elastic) force-versus-displacement relationship.

Link: In an EBF, the segment of a beam that extends from column to brace, located between the end of a diagonal brace and a column, or between the ends of two diagonal braces of the EBF. The length of the link is defined as the clear distance between the diagonal brace and the column face or between the ends of two diagonal braces.

Link intermediate web stiffeners: Vertical web stiffeners placed within the link.

Link rotation angle: The angle of plastic rotation between the link and the beam outside of the link derived using the specified base shear, V.

Liquefaction: An earthquake-induced process in which saturated, loose, granular soils lose a substantial amount of shear strength as a result of increase in pore-water pressure during earthquake shaking.

Load duration: The period of continuous application of a given load, or the cumulative period of intermittent applications of load. (See time effect factor.)

Load path: A path that seismic forces pass through to the foundation of the structure and, ultimately, to the soil. Typically, the load travels from the diaphragm through connections to the vertical lateral-force-resisting elements, and then proceeds to the foundation by way of additional connections.

Load sharing: The load redistribution mechanism among parallel components constrained to deflect together.

Load/slip constant: The ratio of the applied load to a connection and the resulting lateral deformation of the connection in the direction of the applied load.

LRFD (Load and Resistance Factor Design): A method of proportioning structural components (members, connectors, connecting elements, and assemblages) using load and resistance factors such that no applicable limit state is exceeded when the structure is subjected to all design load and resistance factor combinations using load and resistance factors such that no applicable limit state is exceeded when the structure is subjected to all design load combinations.

Lumber: The product of the sawmill and planing mill, usually not further manufactured other than by sawing, resawing, passing lengthwise through a standard planing machine, crosscutting to length, and matching.

Lumber size: Lumber is typically referred to by size classifications. Additionally, lumber is specified by manufacturing classification. Rough lumber and dressed lumber are two of the routinely used manufacturing classifications.

M

Masonry: The assemblage of masonry units, mortar and possibly grout and/or reinforcement. Types of masonry are classified herein with respect to the type of the masonry units such as clay-unit masonry, concrete masonry, or hollow-clay tile masonry.

Mat-formed panel: A structural panel designation representing panels manufactured in a mat-formed process, such as oriented strand board and waferboard.

Maximum Considered Earthquake (MCE): An extreme earthquake hazard level used in the formation of Rehabilitation Objectives. (See BSE-2.)

Maximum displacement: The maximum earthquake displacement of an isolation or energy dissipation system, or elements thereof, excluding additional displacement due to actual or accidental torsion.

Mean return period: The average period of time, in years, between the expected occurrences of an earthquake of specified severity.

Model Building Type: Fifteen common building types used to categorize expected deficiencies, reasonable rehabilitation methods, and estimated costs. See Table 10-2 for descriptions of Model Building Types.

Moisture content: The weight of the water in wood expressed as a percentage of the weight of the oven-dried wood.

Moment frame: A building frame system in which seismic shear forces are resisted by shear and flexure in members and joints of the frame.

N

Narrow wood shear wall: Wood shear walls with an aspect ratio (height to width) greater than two to one. These walls are relatively flexible and thus tend to be incompatible with other building components, thereby taking less shear than would be anticipated when compared to wider walls.

Nominal size: The approximate rough-sawn commercial size by which lumber products are known and sold in the market. Actual rough-sawn sizes vary from the nominal. Reference to standards or grade rules is required to determine nominal to actual finished size relationships, which have changed over time.

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

Nonbearing wall: A wall that supports gravity loads less than as defined for a bearing wall.

Noncompact member: A steel section in compression whose width-to-thickness ratio does not meet the limiting values for compactness, as shown in Table B5.1 of AISC (1986).

Noncomposite masonry wall: Multiwythe masonry wall acting without composite action.

Nonlinear procedure: Analysis based on and including both elastic and post-yield force-versus-displacement relationships.

Nonstructural component: An architectural, mechanical, plumbing, or electrical component, or item of interior equipment and furnishing, permanently installed in the building, as listed in Table 11-1.

Nonstructural Performance Level: A limiting damage state for nonstructural building components used to define Rehabilitation Objectives.

O

Ordinary Moment Frame (OMF): A moment frame system that meets the requirements for Ordinary Moment Frames as defined in seismic provisions for new construction in AISC (1994a), Chapter 5.

Oriented strandboard: A structural panel comprising thin elongated wood strands with surface layers arranged in the long panel direction and core layers arranged in the cross panel direction.

Out-of-plane wall: A wall that resists lateral forces applied normal to its plane.

Overturning: When the moment produced at the base of vertical lateral-force-resisting elements is larger than the resistance provided by the foundation's uplift resistance and building weight.

P

Panel: A sheet-type wood product.

Panel rigidity or stiffness: The in-plane shear rigidity of a panel, the product of panel thickness and modulus of rigidity.

Panel shear: Shear stress acting through the panel thickness.

Panel zone: Area of a column at the beam-to-column connection delineated by beam and column flanges.

Parametric analysis: Repetitive analyses performed in which one or more independent parameters are varied for the ultimate purpose of optimizing a dependent (earthquake response) parameter.

Parapet: Portions of a wall extending above the roof diaphragm. Parapets can be considered as flanges to roof diaphragms if adequate connections exist or are provided.

Partially grouted masonry wall: A masonry wall containing grout in some of the cells.

Particleboard: A panel manufactured from small pieces of wood, hemp, and flax, bonded with synthetic or organic binders, and pressed into flat sheets.

P-Δ effect: Secondary effect of column axial loads and lateral deflection on the shears and moments in various components of a structure.

Perforated wall or infill panel: A wall or panel not meeting the requirements for a solid wall or infill panel.

Pier: Similar to pile; usually constructed of concrete and cast in place.

Pile: A deep structural component transferring the weight of a building to the foundation soils and resisting vertical and lateral loads; constructed of concrete, steel, or wood; usually driven into soft or loose soils.

Pitch or spacing: The longitudinal center-to-center distance between any two consecutive holes or fasteners in a row.

Plan irregularity: Horizontal irregularity in the layout of vertical lateral-force-resisting elements, thus producing a differential between the center of mass and center of rigidity, that typically results in significant torsional demands on the structure.

Planar shear: The shear that occurs in a plane parallel to the surface of a panel, which has the ability to cause the panel to fail along the plies in a plywood panel or in a random layer in a nonveneer or composite panel.

Platform framing: Construction method in which stud walls are constructed one floor at a time, with a floor or roof joist bearing on top of the wall framing at each level.

Ply: A single sheet of veneer, or several strips laid with adjoining edges that form one veneer lamina in a glued plywood panel.

Plywood: A structural panel comprising plies of wood veneer arranged in cross-aligned layers. The plies are bonded with an adhesive that cures upon application of heat and pressure.

Pole: A round timber of any size or length, usually used with the larger end in the ground.

Pole structure: A structure framed with generally round continuous poles that provide the primary vertical frame and lateral-load-resisting system.

Pounding: Two adjacent buildings coming in contact during earthquake excitation because they are too close together and/or exhibit different dynamic deflection characteristics.

Prescriptive ultimate bearing capacity: Assumption of ultimate bearing capacity based on properties prescribed in Section 4.4.1.2.

Preservative: A chemical that, when suitably applied to wood, makes the wood resistant to attack by fungi, insects, marine borers, or weather conditions.

Pressure-preservative treated wood: Wood products pressure-treated by an approved process and preservative.

Presumptive ultimate bearing capacity: Assumption of ultimate bearing capacity based on allowable loads from original design.

Primary (strong) panel axis: The direction that coincides with the length of the panel.

Primary component: Those components that are required as part of the building's lateral-force-resisting system (as contrasted to secondary components).

Primary element: An element that is essential to the ability of the structure to resist earthquake-induced deformations.

Punched metal plate: A light steel plate fastening having punched teeth of various shapes and configurations that are pressed into wood members to effect transfer shear. Used with structural lumber assemblies.

R

Redundancy: Quality of having alternative paths in the structure by which the lateral forces are resisted, allowing the structure to remain stable following the failure of any single element.

Re-entrant corner: Plan irregularity in a diaphragm, such as an extending wing, plan inset, or E-, T-, X-, or L-shaped configuration, where large tensile and compressive forces can develop.

Rehabilitation Method: A procedural methodology for the reduction of building earthquake vulnerability.

Rehabilitation Objective: A statement of the desired limits of damage or loss for a given seismic demand, which is usually selected by the owner, engineer, and/or relevant public agencies. (See Chapter 2.)

Rehabilitation strategy: A technical approach for developing rehabilitation measures for a building to reduce its earthquake vulnerability.

Reinforced masonry (RM) wall: A masonry wall that is reinforced in both the vertical and horizontal directions. The sum of the areas of horizontal and vertical reinforcement must be at least 0.002 times the gross cross-sectional area of the wall, and the minimum area of reinforcement in each direction must be not less than 0.0007 times the gross cross-sectional area of the wall. Reinforced walls are assumed to resist loads through resistance of the masonry in compression and the reinforcing steel in tension or compression. Reinforced masonry is partially grouted or fully grouted.

Repointing: A method of repairing a cracked or deteriorating mortar joint in masonry. The damaged or deteriorated mortar is removed and the joint is refilled with new mortar.

Required member resistance: Load effect (force, moment, stress, action as appropriate) acting on an element or connection, determined by structural analysis from the factored loads and the critical load combinations.

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

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

Resistance factor: A reduction factor applied to member resistance that accounts for unavoidable deviations of the actual strength from the nominal value, and the manner and consequences of failure.

Retaining wall: A free-standing wall that has soil on one side.

Rigid diaphragm: A diaphragm that meets requirements of Section 3.2.4

Rough lumber: Lumber as it comes from the saw prior to any dressing operation.

Row of fasteners: Two or more fasteners aligned with the direction of load.

Running bond: A pattern of masonry where the head joints are staggered between adjacent courses by more than a third of the length of a masonry unit. Also refers to the placement of masonry units such that head joints in successive courses are horizontally offset at least one-quarter the unit length.

S

Seasoned lumber: Lumber that has been dried. Seasoning takes place by open-air drying within the limits of moisture contents attainable by this method, or by controlled air drying (i.e., kiln drying).

Secondary component: Those components that are not required for lateral force resistance (contrasted to Primary Components). They may or may not actually resist some lateral forces.

Secondary component: Those components that are not required for lateral force resistance (contrasted to primary components). They may or may not actually resist some lateral forces.

Secondary element: An element that does not affect the ability of the structure to resist earthquake-induced deformations.

Seismic demand: Seismic hazard level commonly expressed in the form of a ground shaking response spectrum. It may also include an estimate of permanent ground deformation.

Shallow foundation: Isolated or continuous spread footings or mats.

Shear wall: A wall that resists lateral forces applied parallel with its plane. Also known as an in-plane wall.

Sheathing: Lumber or panel products that are attached to parallel framing members, typically forming wall, floor, ceiling, or roof surfaces.

Short captive column: Columns with height-to-depth ratios less than 75% of the nominal height-to-depth ratios of the typical columns at that level. These columns, which may not be designed as part of the primary lateral-load-resisting system, tend to attract shear forces because of their high stiffness relative to adjacent elements.

Shrinkage: Reduction in the dimensions of wood due to a decrease of moisture content.

Simplified Rehabilitation Method: An approach, applicable to some types of buildings and Rehabilitation Objectives, in which analyses of the entire building's response to earthquake hazards are not required.

Slip-critical joint: A bolted joint in which slip resistance of the connection is required.

Solid masonry unit: A masonry unit whose net cross-sectional area in every plane parallel to the bearing surface is 75% or more of the gross cross-sectional area in the same plane.

Solid wall or solid infill panel: A wall or infill panel with openings not exceeding 5% of the wall surface area. The maximum length or height of an opening in a solid wall must not exceed 10% of the wall width or story height. Openings in a solid wall or infill panel must be located within the middle 50% of a wall length and story height, and must not be contiguous with adjacent openings.

Special Moment Frame (SMF): A moment frame system that meets the special requirements for frames as defined in seismic provisions for new construction.

SPT N-Values: Using a standard penetration test (ASTM Test D1586), the number of blows of a 140-pound hammer falling 30 inches required to drive a standard 2-inch-diameter sampler a distance of 12 inches.

Stack bond: In contrast to running bond, usually a placement of units such that the head joints in successive courses are aligned vertically.

Stiff diaphragm: A diaphragm that meets requirements of Section 3.2.4.

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

Strength: The maximum axial force, shear force, or moment that can be resisted by a component.

Stress resultant: The net axial force, shear, or bending moment imposed on a cross section of a structural component.

Strong back system: A secondary system, such as a frame, commonly used to provide out-of-plane support for an unreinforced or under-reinforced masonry wall.

Strong column-weak beam: The capacity of the column at any moment frame joint must be greater than those of the beams, in order to ensure inelastic action in the beams, thereby localizing damage and controlling drift.

Structural Performance Level: A limiting structural damage state, used in the definition of Rehabilitation Objectives.

Structural Performance Range: A range of structural damage states, used in the definition of Rehabilitation Objectives.

Structural system: An assemblage of load-carrying components that are joined together to provide regular interaction or interdependence.

Structural-use panel: A wood-based panel product bonded with an exterior adhesive, generally 4' x 8' or larger in size. Included under this designation are plywood, oriented strand board, waferboard, and composite panels. These panel products meet the requirements of PS 1-83 or PS 2-92 and are intended for structural use in residential, commercial, and industrial applications.

Stud: Wood member used as vertical framing member in interior or exterior walls of a building, usually 2" x 4" or 2" x 6" sizes, and precision end-trimmed.

Subassembly: A portion of an assembly.

Subdiaphragm: A portion of a larger diaphragm used to distribute loads between members.

Systematic Rehabilitation Method: An approach to rehabilitation in which complete analysis of the building's response to earthquake shaking is performed.

T

Target displacement: An estimate of the likely building roof displacement in the design earthquake.

Tie: See **drag strut.**

Tie-down: Hardware used to anchor the vertical chord forces to the foundation or framing of the structure in order to resist overturning of the wall.

Tie-down system: The collection of structural connections, components, and elements that provide restraint against uplift of the structure above the isolation system.

Timbers: Lumber of nominal five or more inches in smaller cross-section dimension.

Time effect factor: A factor applied to adjusted resistance to account for effects of duration of load. (See load duration.)

Total design displacement: The BSE-1 displacement of an isolation or energy dissipation system, or elements thereof, including additional displacement due to actual and accidental torsion.

Total maximum displacement: The maximum earthquake displacement of an isolation or energy dissipation system, or elements thereof, including additional displacement due to actual and accidental torsion.

Transverse wall: A wall that is oriented transverse to the in-plane shear walls, and resists lateral forces applied normal to its plane. Also known as an out-of-plane wall.

U

Ultimate bearing capacity: Maximum possible foundation load or stress (strength); increase in deformation or strain results in no increase in load or stress.

Unreinforced masonry (URM) wall: A masonry wall containing less than the minimum amounts of reinforcement as defined for masonry (RM) walls. An unreinforced wall is assumed to resist gravity and lateral loads solely through resistance of the masonry materials.

V

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

Velocity-dependent energy dissipation devices (EDDs): Devices having mechanical characteristics such that the force in the device is dependent on the relative velocity in the device.

Vertical irregularity: A discontinuity of strength, stiffness, geometry, or mass in one story with respect to adjacent stories.

W

Waferboard: A nonveneered structural panel manufactured from two- to three-inch flakes or wafers bonded together with a phenolic resin and pressed into sheet panels.

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.

Wythe: A continuous vertical section of a wall, one masonry unit in thickness.

X

X-braced frame: A concentric braced frame (CBF) in which a pair of diagonal braces crosses near the mid-length of the braces.

Y

Y-braced frame: An eccentric braced frame (EBF) in which the stem of the Y is the link of the EBF system.

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	To Convert	To	Multiply By
LENGTH	inches (in.)	millimeters (mm)	25.4
		meters (m)	0.0254
	feet (ft)	millimeters (mm)	304.8
		meters (m)	0.3048
AREA	square inches (in.^{2})	square millimeters (mm^2)	645.16
		square meters (m^2)	0.00064516
	square feet (ft^2)	square millimeters (mm^2)	92903
		square meters (m^2)	0.092903
FORCE	pounds (lb)	newtons (N)	4.4482
		kilonewtons (kN)	0.004482
	kips (k)	newtons (N)	4448.2
		kilonewtons (kN)	4.4482
FORCE LENGTH (BENDING MOMENT, TORQUE)	inch-pounds (in.-lb)	newton-millimeters (N-mm)	112.98
		newton-meters (N-m)	0.11298
	foot-pounds (ft-lb)	newton-millimeters (N-mm)	1355.8
		newton-meters (N-m)	1.3558
	inch-kips (in.-k)	kilonewton-millimeters (kN-mm)	112.98
		kilonewton-meters (kN-m)	0.11298
	foot-kips (ft-k)	kilonewton-millimeters (kN-mm)	1355.8
		kilonewton-meters (kN-m)	1.3558
FORCE/LENGTH	pounds/inch (lb/in.)	newtons/millimeter (N-mm)	0.17513
		newtons/meter (N-m)	175.13
	pounds/foot (lb/ft)	newtons/millimeter (N-mm)	0.014594
		newtons/meter (N-m)	14.594
	kips/inch (k/in.)	kilonewtons/millimeter (kN-mm)	0.17513
		kilonewtons/meter (kN-m)	175.13
	kips/foot (k/ft)	kilonewtons/millimeter (kN-mm)	0.014594
		kilonewtons/meter (kN-m)	14.594
FORCE/AREA (MODULUS, PRESSURE, STRESS)	pounds/inch^2 (lb/in.^{2})	pascals (Pa)	6894.8
		kilopascals (kPa)	6.8948
	pounds/foot^2 (lb/ft^2)	pascals (Pa)	47.88
		kilopascals (kPa)	0.04788
	kips/inch^2 (k/in.^{2})	pascals (Pa)	6894800
		kilopascals (kPa)	6894.8
	kips/foot^2 (k/ft^2)	pascals (Pa)	47880
		kilopascals (kPa)	47.88

US CUSTOMARY TO SI UNIT CONVERSION TABLES

To Convert	To	Multiply By

Note: 1.0 Pa = 1.0 N/m^2