FEDERAL EMERGENCY MANAGEMENT AGENCY FEMA 274 /October 1997

NEHRP COMMENTARY ON THE 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.

BSSC
Seismic
Rehabilitation
Project

NEHRP COMMENTARY ON THE GUIDELINES FOR THE SEISMIC REHABILITATION OF BUILDINGS
(FEMA PUBLICATION 274)

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

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

With funding from the
FEDERAL EMERGENCY MANAGEMENT AGENCY
Washington, D.C.

October 1997
Washington, D.C.

NOTICE: This report was prepared under Cooperative Agreement EMW-91-K-3602 between the Federal Emergency Management Agency and the National Institute of Building Sciences.

For further information concerning this document or the activities of the BSSC, contact the Executive Director, Building Seismic Safety Council, 1090 Vermont Ave., N.W., Suite 700, Washington, D.C. 20005; phone 202-289-7800; fax 202-289-1092; e-mail bssc@nibs.org.

PARTICIPANTS

PROJECT OVERSIGHT COMMITTEE

Eugene Zeller, Chairman
Thomas G. Atkinson, ATC
Gerald Jones, BSSC
Christopher Rojahn, ATC
Paul Seaburg, ASCE
Ashvin Shah, ASCE
James R. Smith, BSSC

BUILDING SEISMIC SAFETY COUNCIL

PROJECT MANAGER

James R. Smith

DEPUTY PROJECT MANAGER

Thomas Hollenbach

TECHNICAL WRITER-EDITOR

Claret Heider

SEISMIC REHABILITATION ADVISORY PANEL

Gerald Jones, Chairman
David Allen
John Battles
David Breiholz
Michael Caldwell
Gregory L. F. Chiu
Terry Dooley
Susan Dowty
Steven J. Eder
S. K. Ghosh
Barry J. Goodno
Charles G. Gutberlet
Warner Howe
Howard Kunreuther
Harry W. Martin
Robert McCluer
Margaret Pepin-Donat
William Petak
Howard Simpson
William Stewart
James Thomas
L. Thomas Tobin

PROJECT COMMITTEE

Warner Howe, Chairman
Gerald H. Jones
Allan R. Porush
F. Robert Preece
William W. Stewart

SOCIETAL ISSUES

Robert A. Olson

FEDERAL EMERGENCY MANAGEMENT AGENCY

PROJECT OFFICER

Ugo Morelli

TECHNICAL ADVISOR

Diana Todd

APPLIED TECHNOLOGY COUNCIL

PRINCIPAL INVESTIGATOR

Christopher Rojahn

PROJECT DIRECTOR

Daniel Shapiro

CO-PROJECT DIRECTOR

Lawrence D. Reaveley

SENIOR TECHNICAL ADVISOR

William T. Holmes

TECHNICAL ADVISOR

Jack P. Moehle

ATC BOARD REPRESENTATIVE

Thomas G. Atkinson

GENERAL REQUIREMENTS

Ronald O. Hamburger, Team Leader
Sigmund A. Freeman
Peter Gergely (deceased)
Richard A. Parmelee
Allan R. Porush

MODELING AND ANALYSIS

Mike Mehrain, Team Leader
Ronald P. Gallagher
Helmut Krawinkler
Guy J. P. Nordenson
Maurice S. Power
Andrew S. Whittaker

GEOTECHNICAL & FOUNDATIONS

Jeffrey R. Keaton, Team Leader
Craig D. Comartin
Paul W. Grant
Geoffrey R. Martin
Maurice S. Power

CONCRETE

Jack P. Moehle, Co-Team Leader
Lawrence D. Reaveley, Co-Team Leader
James E. Carpenter
Jacob Grossman
Paul A. Murray
Joseph P. Nicoletti
Kent B. Soelberg
James K. Wight

MASONRY

Daniel P. Abrams, Team Leader
Samy A. Adham
Gregory R. Kingsley
Onder Kustu
John C. Theiss

STEEL

Douglas A. Foutch, Team Leader
Navin R. Amin
James O. Malley
Charles W. Roeder
Thomas Z. Scarangello

WOOD

John M. Coil, Team Leader
Jeffery T. Miller
Robin Shepherd
William B. Vaughn

NEW TECHNOLOGIES

Charles A. Kircher, Team Leader
Michael C. Constantinou
Andrew S. Whittaker

NONSTRUCTURAL

Christopher Arnold, Team Leader
Richard L. Hess
Frank E. McClure
Todd W. Perbix

SIMPLIFIED REHABILITATION

Chris D. Poland, Team Leader
Leo E. Argiris
Thomas F. Heausler
Evan Reis
Tony Tschanz

QUALIFICATION OF IN-PLACE MATERIALS

Charles J. Hookham, Lead Consultant
Richard Atkinson (deceased)
Ross Esfandiari

LANGUAGE & FORMAT

James R. Harris

REPORT PREPARATION

Roger E. Scholl (deceased), Lead Consultant
Robert K. Reitherman
A. Gerald Brady, Copy Editor
Patty Christofferson, Coordinator
Peter N. Mork, Illustrations

AMERICAN SOCIETY OF CIVIL ENGINEERS

REHABILITATION STEERING COMMITTEE

Vitelmo V. Bertero
Paul Seaburg
Roland L. Sharpe
Jon S. Traw
Clarkson W. Pinkham
William J. Hall

USERS WORKSHOPS

Tom McLane, Manager
Debbie Smith, Coordinator

RESEARCH SYNTHESIS

James O. Jirsa

SPECIAL ISSUES

Melvyn Green

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

Table of Contents

List of Figures

List of Tables

C1. No Commentary for Chapter 1

C2. General Requirements (Simplified and Systematic Rehabilitation)

C2.1 Scope

No commentary is provided for this section.

C2.2 Basic Approach

The basic steps that the rehabilitation design process comprises are indicated in this section. Prior to embarking on a rehabilitation design, it is necessary to understand whether the building, in its existing condition, is capable of meeting the intended Performance Levels. This requires that a preliminary evaluation of the building be performed. BSSC (1992a) is indicated as one potential guideline for performing such evaluations; however, it is noted that BSSC (1992a) does not directly address many of the Rehabilitation Objectives that are included within the scope of this document. One possible approach to performing a preliminary evaluation, in order to determine if rehabilitation is necessary to meet other Rehabilitation Objectives, would be to analyze the building, without corrective measures, using the methods contained in this document.

An important step in the design of rehabilitation measures is the development of a preliminary design. While the *Guidelines* provide information on alternative rehabilitation strategies that could be employed, they do not provide a direct methodology for arriving at a preliminary design. The general approach recommended is one of examining the deficiencies in the existing structure—relative to the acceptance criteria provided in the *Guidelines* for the desired Performance Level—in order to determine the principal requirements for additional strength, stiffness, or deformation capacity. A strategy should be selected that addresses these requirements in an efficient manner. Preliminary design must be made largely by trial and error, relying heavily on the judgment of the design engineer.

C2.3 Design Basis

The *Guidelines* provide uniform criteria by which existing buildings may be rehabilitated to attain a wide range of different Performance Levels, when subjected to earthquakes of varying severities and probability of occurrence. This is a unique approach, distinctly different from that presently adopted by building codes for new construction. In the building codes for new construction, building performance is implicitly set in a manner that is not transparent to the user. Therefore, the user frequently does not understand the level of performance to be expected of buildings designed to the code, should they experience a design event. Further, the user is not given a clear understanding of what design changes should be made in order to obtain performance different from that implicit in the codes. The *Guidelines* start by requiring that the user select specific performance goals, termed Rehabilitation Objectives, as a basis for design. In this way, users can directly determine the effect of different performance goals on the design requirements.

It is important to note that when an earthquake does occur, there can be considerable variation in the levels of performance experienced by similar buildings located on the same site, and therefore apparently subjected to the same earthquake demands. This variability can result from a number of factors, including random differences in the levels of workmanship, material strength, and condition of each structure, the amount and distribution of live load present at the time of the earthquake, the influence of nonstructural components present within each structure, the response of the soils beneath the buildings, and relatively minor differences in the character of the ground motion transmitted to the structures. Many of these factors cannot be completely identified or quantified at our current level of understanding and capability.

It is the intent of the *Guidelines* that most, although not necessarily all, structures designed to attain a given performance at a specific earthquake demand would exhibit behavior superior to that predicted. However, there is no guarantee of this. There is a finite possibility that—as a result of the variances described above, and other factors—some rehabilitated buildings would experience poorer behavior than that intended by the Rehabilitation Objective.

The concept of redundancy is extremely important to the design of structures for seismic resistance, in that it is expected that significant damage to the structural elements can occur as a result of building response to severe ground motion. In a redundant structure, multiple elements (or components) will be available to resist forces induced by such response. Should one or more of these elements fail, or become so badly damaged that they are no longer effective in providing structural resistance, additional elements are available to prevent loss of stability. In a nonredundant structure, failure of one or two elements can result in complete loss of lateral resistance, and collapse.

In many structures, nearly all elements and components of the building participate in the structure's lateral-load-resisting system, to some extent. As the structure is subjected to increasing lateral demands, some of these elements may begin to fail and lose strength much sooner than others. If a structure has sufficient redundancy, it may be permissible to allow failure of some of these elements, as long as this does not result in loss of gravity load-carrying capacity or overall lateral stability. The *Guidelines* introduce the concept of "primary" and "secondary" elements in order to allow designers to take advantage of the inherent redundancy in some structures, and to permit a few selected elements of the structure to experience excessive damage rather than requiring massive rehabilitation programs to prevent such damage.

Any element in a structure may be designated as a secondary element, so long as expected damage to the element does not compromise the ability of the structure to meet the intended performance levels. Secondary elements are assumed to have minimal effective contribution to the lateral-force-resisting system. When linear analysis procedures are used, secondary elements are not typically modeled as part of the system, or if they are, they are modeled at greatly reduced stiffnesses, simulating their anticipated stiffness degradation under large lateral response. Primary elements must remain effective in resisting lateral forces, in order to provide the basic stability of the structure.

For some structures, it may be possible to determine at the beginning of the design process which elements should be classified as primary or secondary. For other, more complex structures, it may be necessary to perform initial evaluations assuming all elements are primary. If some of the elements cannot meet the applicable acceptance criteria, or have demands that exceed their acceptance criteria by substantially greater margins then other elements, these could be designated as secondary, and the analysis repeated with the model altered to remove the stiffness contribution of these elements. If too many elements are designated as secondary, the structure's ability to resist the required demands will be impaired, indicating that additional rehabilitation measures are required.

C2.4 Rehabilitation Objectives

The Rehabilitation Objective(s) selected for a project are an expression of the desired building behavior when it experiences earthquake effects of projected severity. In the *Guidelines*, selection of a Rehabilitation Objective controls nearly all facets of the design process, including the characterization of earthquake demands, the analytical techniques that may be used to predict building response to these demands, and the acceptance criteria (strength and deformability parameters) used to judge the design's adequacy.

In the *NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings: 1994 Edition* (BSSC, 1995), three different design performance objectives are implicitly set, based on the building's intended occupancy. Most buildings are contained within Seismic Hazard Exposure Group I, for which a basic design objective of minimizing the hazard to life safety is adopted. For high-occupancy buildings, contained in Seismic Hazard Exposure Group II, the same performance objective is set, but with a higher degree of reliability. Buildings that contain occupancies essential to post-disaster response are grouped within Seismic Hazard Exposure Group III, for which a design objective of post-earthquake functionality is set. The Seismic Hazard Exposure Group together with the site seismicity determine the building's Seismic Performance Category and, therefore, the permissible structural systems, the analytical procedures that may be employed, the types of structural detailing that must be incorporated, and the design requirements for nonstructural components.

In the formation of the *Guidelines*, it was felt that a rigid requirement to upgrade all buildings to the performance objective corresponding with their Seismic Hazard Exposure Group in the *NEHRP Recommended Provisions* would be prohibitively expensive; could result in extensive demolition of structures that are valuable cultural, societal and historic resources; or alternatively, would achieve no improvement in the public safety, through a lack of implementation. It was also recognized that there are a number of owners who desire better seismic performance for individual structures than is provided for in the corresponding

Seismic Hazard Exposure Group of the BSSC (1995) provisions. Therefore, the *Guidelines* adopt a flexible approach with regard to selection of Rehabilitation Objectives. For each building, a decision must be made as to the acceptable behavior for different levels of seismic hazard, balanced with the cost of rehabilitating the structure to obtain that behavior. For many buildings, multiple rehabilitation objectives will be adopted—ranging from negligible damage and occupancy interruption for earthquake events with a high probability of occurrence, to substantial damage but protection of life safety for events with a low probability of occurrence. Figure C2-1 summarizes the various Rehabilitation Objectives available to users of the *Guidelines*. BSE-1 is the Basic Safety Earthquake 1; BSE-2, the more severe ground motion defined with regard to the Basic Safety Objective (BSO), is Basic Safety Earthquake 2.

In general, Rehabilitation Objectives that expect relatively low levels of damage for relatively infrequent earthquake events will result in more extensive rehabilitation work and greater expense than objectives with more modest goals of controlling damage. Figure C2-2 schematically presents the relationship between different Rehabilitation Objectives and probable program cost. VSP (1992), *A Benefit-Cost Model for the Seismic Rehabilitation of Buildings* provides a methodology for evaluation of the costs and benefits of seismic rehabilitation.

The formation of project Rehabilitation Objectives requires the selection of both the target Building Performance Levels and the corresponding earthquake hazard levels for which they are to be achieved. Hazard levels may be selected on either a probabilistic or deterministic basis and may be selected at any level of severity. This is also a significant departure from the practice adopted in building codes for new construction.

C2.4.1 Basic Safety Objective

Rehabilitation design for the Basic Safety Objective (BSO) under the *Guidelines* is expected to produce earthquake performance similar—but not identical—to that desired for new buildings in Seismic Hazard Exposure Group I of BSSC (1995). Buildings that are rehabilitated for the BSO will in general present a low level of risk to life safety at any earthquake demand level likely to affect them. However, some potential for life safety endangerment at the extreme levels of demand that can occur at the site will remain. In addition, buildings rehabilitated to these Performance Levels may also have significant potential for extreme damage and total economic loss when subjected to relatively infrequent but severe earthquake events. To the extent that it is economically feasible, all buildings should be rehabilitated to meet this objective, as a minimum.

The *Guidelines* specify a two-level design check (Life Safety Performance Level for BSE-1 demands and Collapse Prevention Performance Level for BSE-2 demands) in order to design for the BSO. This is in contrast to the BSSC (1995) provisions, which employ only a single level design check. The BSSC (1995) provisions can adopt the single level design approach because for new structures it is possible to control the ductility and configuration of the design to an extent that will permit those structures designed to achieve the Life Safety Performance Level for a 10%/50 year event to also avoid collapse for much larger events. Existing buildings have not generally been constructed with the same controls on configuration and detailing, and therefore may not have comparable capacity to survive stronger earthquake demands, even when rehabilitated. Therefore, it was considered prudent to explicitly require evaluation of the rehabilitated structure for its capacity to resist collapse when subjected to extreme earthquake demands.

The *Guidelines* permit individual building officials to declare, or deem, that buildings in compliance with the 1994 or later editions of the *Uniform Building Code* (ICBO, 1994) or *Standard Building Code* (SBCCI, 1994), or with the 1993 edition of the *National Building Code* (BOCA, 1993) meet the requirements of the BSO. This was done recognizing that the *Guidelines* represent new technology which would in some cases provide different results than would the provisions of current model codes, and to avoid the problem of creating a class of hazardous buildings comprising newly constructed, code-compliant structures. Buildings that have been adequately designed and constructed in conformance with the provisions of the 1994 *Uniform Building Code* for seismic zones 3 and 4, or with the provisions of the 1993 *National Building Code* or 1994 *Standard Building Code* for Seismic Performance Categories D or E, should, in actuality, meet or exceed the BSO. However, buildings designed for lower seismic zones or performance categories, or that have not been adequately designed and constructed in conformance with the code provisions, may not be able to meet the technical requirements or performance expectations of the BSO. It is anticipated that buildings

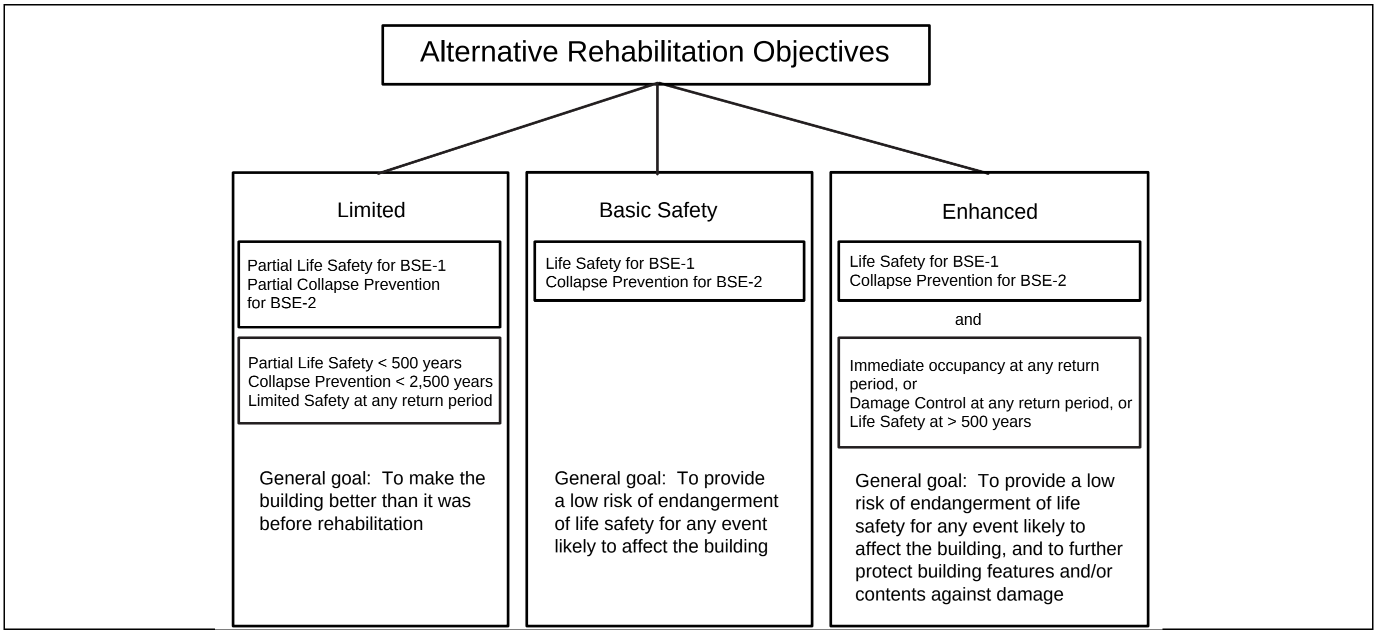


Figure C2-1 Rehabilitation Objectives

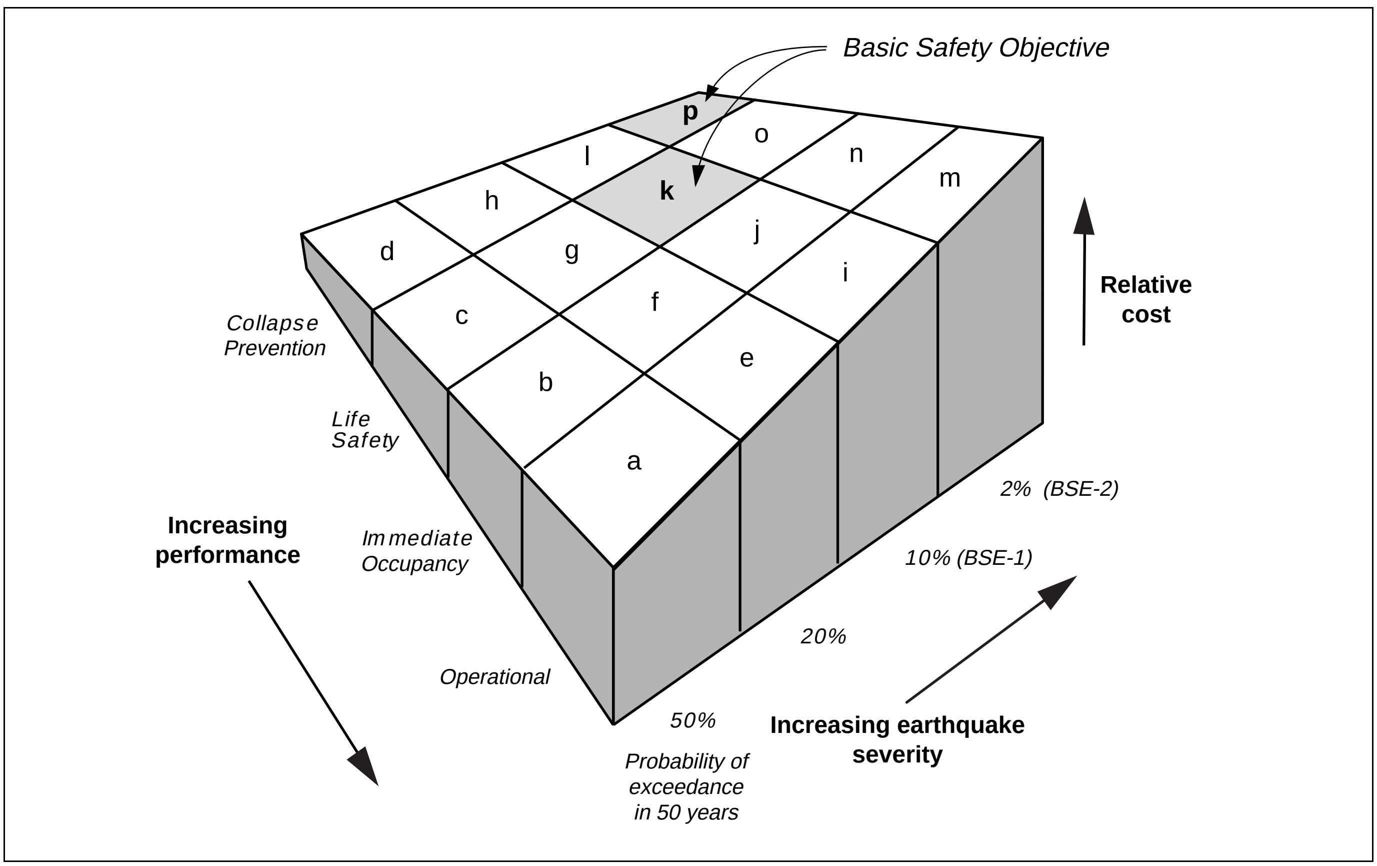


Figure C2-2 Surface Showing Relative Costs of Various Rehabilitation Objectives

meeting code provisions based on seismic design criteria contained in the *NEHRP Recommended Provisions* (BSSC, 1997) would be able to meet or exceed the BSO regardless of the seismic zone or performance category ("Seismic Design Category" in the 1997 *NEHRP Provisions*) for which they have been designed.

C2.4.2 Enhanced Rehabilitation Objectives

Individual agencies and owners may elect to design to Rehabilitation Objectives that provide for lower levels of damage than anticipated for buildings rehabilitated to the BSO. Benefits of such rehabilitation are potential reductions of damage repair costs and loss of facility use, as well as greater confidence in the protection of life safety.

There are many buildings for which the levels of damage that may be sustained under the BSO will be deemed inappropriate. These may include buildings in NEHRP Seismic Hazard Exposure Group III as defined in the 1994 *NEHRP Provisions* (BSSC, 1995)—such as hospitals, fire stations, and similar facilities critical to post-earthquake disaster response and recovery—as well as buildings housing functions critical to the economic welfare of business concerns, such as data processing centers and critical manufacturing facilities. It may be desirable that such buildings be available to perform their basic functions shortly after an earthquake occurs. Designing to the Immediate Occupancy Performance Level, or to a custom level within the Damage Control Performance Range, at an appropriate earthquake hazard level, provides an opportunity to achieve such performance.

The importance of maintaining operations or controlling damage within an individual building should be considered in selecting an appropriate Rehabilitation Objective to use in the rehabilitation design. For buildings in NEHRP Seismic Hazard Exposure Group III, Performance Levels consisting of Immediate Occupancy for BSE-1 and Life Safety for BSE-2 demands could be considered as a basis for design. Buildings designed to such objectives will in general present a low level of risk that the buildings could not be occupied at any earthquake demand level likely to affect them, and a very low risk of life safety endangerment. However, it is not intended that structures designed to these Rehabilitation Objectives would behave so well that no interruption in their service occurs. Some cleanup and repair may be required in order to restore such structures to service; however, it is intended that such activities can be quickly accomplished.

For buildings contained in NEHRP Seismic Hazard Exposure Group II, and for buildings in critical business occupancies, Rehabilitation Objectives consisting of Damage Control Performance Range for 10%/50 year earthquake demands and Life Safety Performance Level for MCE demands should be considered. Buildings rehabilitated to such objectives would have a low level of risk of long-term occupancy interruption resulting from earthquake damage, as well as a very low level of risk of life safety endangerment.

It is important to note that mere provision of structural integrity does not ensure that buildings housing critical functions will be operable immediately following an earthquake. In addition to damage control, functionality following an earthquake typically requires electric power, as well as other utilities. Facilities that must remain in service in the immediate post-earthquake period should be provided with reliable standby utilities to service their essential systems. In addition, critical equipment within the facilities should be safeguarded to ensure functionality. Discussions of these requirements are contained in Chapter 11 on nonstructural components.

The determination as to whether a project should be designed to Enhanced Rehabilitation Objectives, and if so, which Performance Levels should be coupled with which earthquake demand levels, largely depends on the acceptable level of risk for the facility. Cost-benefit analysis may be a useful tool for establishing an appropriate Enhanced Rehabilitation Objective for many facilities.

C2.4.3 Limited Rehabilitation Objectives

Limited Rehabilitation provides for seismic rehabilitation to reliability levels that are lower than the BSO. It is included in the *Guidelines* to provide a method for owners and agencies with limited economic resources to obtain a reduction in their existing seismic risk, rather than doing nothing. Rehabilitation to objectives that do not meet the BSO may be selected by individual agencies or owners when it is deemed economically impractical to design for the BSO. The usual intent of such rehabilitation is to achieve highly cost-effective improvement in the probable earthquake performance of the building. Two types of Limited Rehabilitation Objectives are included.

C2.4.3.1 Partial Rehabilitation

Partial Rehabilitation is rehabilitation that addresses only a portion of the building. The typical goal of Partial Rehabilitation is to reduce the specific risks related to one or more common or particularly severe vulnerabilities, without addressing the building's complete lateral-force-resisting system or all nonstructural components. It is recommended that Partial Rehabilitation Objectives be identical to those for the BSO. In this way, partial rehabilitation may be implemented as one of a series of incremental rehabilitation measures that, when taken together, achieve full rehabilitation of the building to the BSO. Alternatively, other Rehabilitation Objectives could be selected as the basis for partial rehabilitation.

C2.4.3.2 Reduced Rehabilitation

Reduced Rehabilitation Objectives address the entire structure; however, they permit greater levels of damage, at more probable levels of ground motion, than is permitted under the BSO. Reduced Rehabilitation Objectives permit owners with limited resources to reduce the levels of damage in the more moderate events likely to occur with relative frequency over the building's life. These objectives may be most appropriate for buildings with limited remaining years of life or with relatively low or infrequent occupancies.

C2.5 Performance Levels

Building performance in these *Guidelines* is expressed in terms of Building Performance Levels. These Building Performance Levels are discrete damage states selected from among the infinite spectrum of possible damage states that buildings could experience as a result of earthquake response. The particular damage states identified as Building Performance Levels in these *Guidelines* have been selected because these Performance Levels have readily identifiable consequences associated with the post-earthquake disposition of the building that are meaningful to the building user community. These include the ability to resume normal functions within the building, the advisability of post-earthquake occupancy, and the risk to life safety.

Although a building's performance is a function of the performance of both structural systems and nonstructural components and contents, these are treated independently in the *Guidelines*, with separate Structural and Nonstructural Performance Levels defined. Each Building Performance Level comprises the individual Structural and Nonstructural Performance Levels selected by the design team. This subcategorization of building performance into separate structural and nonstructural components was adopted in the *Guidelines* because building owners have frequently approached building rehabilitation projects in this manner. Historically, many building owners have performed seismic rehabilitation projects that concentrated effort in the improvement of the structural performance capability of the building without addressing nonstructural vulnerabilities. Such owners typically believed that if the building performance could be controlled to provide limited levels of structural damage, damage to nonstructural components could be dealt with in an acceptable manner. Many other owners have taken a directly contrary approach, believing that it was most important to prevent damage to nonstructural building components, since such components have often been damaged in even relatively moderate earthquakes, resulting in costly business interruption. The approach taken by the *Guidelines* provides sufficient flexibility to accommodate either approach to building rehabilitation, as well as approaches that address structural and nonstructural vulnerabilities in a more balanced manner.

C2.5.1 Structural Performance Levels and Ranges

When a building is subjected to earthquake ground motion, a pattern of lateral deformations that varies with time is induced into the structure. At any given point in time, a particular state of lateral deformation will exist in the structure, and at some time within the period in which the structure is responding to the ground motion, a maximum pattern of deformation will occur. At relatively low levels of ground motion, the deformations induced within the building will be limited, and the resulting stresses that develop within the structural components will be within the elastic range of behavior. Within this elastic range, the structure will experience no damage. All structural components will retain their original strength, stiffness, and appearance, and when the ground motion stops, the structure will return to its pre-earthquake condition.

At more severe levels of ground motion, the lateral deformations induced into the structure will be larger. As these deformations increase, so will demands on the individual structural components. At different levels of deformation, corresponding to different levels of ground motion severity, individual components of the structure will be strained beyond their elastic range. As this occurs, the structure starts to experience damage in the form of cracking, spalling, buckling, and yielding of the various components. As components become damaged, they degrade in stiffness, and some elements will begin to lose their strength. In general, when a structure has responded to ground motion within this range of behavior, it will not return to its pre-earthquake condition when the ground motion stops. Some permanent deformation may remain within the structure and damage will be evident throughout. Depending on how far the structure has been deformed, and in what pattern, the structure may have lost a significant amount of its original stiffness and, possibly, strength.

Brittle elements are not able to sustain inelastic deformations and will fail suddenly; the consequences may range from local and repairable damage to collapse of the structural system. At higher levels of ground motion, the lateral deformations induced into the structure will strain a number of elements to a point at which the elements behave in a brittle manner or, as a result of the decreased overall stiffness, the structure loses stability. Eventually, partial or total collapse of the structure can occur. The Structural Performance Levels and Ranges used in the *Guidelines* relate the extent of a building's response to earthquake hazards to these various possible damage states.

Figure C2-3 illustrates the behavior of a ductile structure as it responds with increasing lateral deformation. The figure is a schematic plot of the lateral force induced in the structure as a function of lateral deformation. Three discrete points are indicated, representing the discrete Performance Levels: Immediate Occupancy, Life Safety, and Collapse Prevention.

At the Immediate Occupancy Level, damage is relatively limited. The structure retains a significant portion of its original stiffness and most if not all of its strength. At the Collapse Prevention Level, the building has experienced extreme damage. If laterally deformed beyond this point, the structure can experience instability and collapse. At the Life Safety Level, substantial damage has occurred to the structure, and it may have lost a significant amount of its original stiffness. However, a substantial margin remains for additional lateral deformation before collapse would occur.

Specifically, it is intended that structures meeting the Life Safety Level would be able to experience at least 33% greater lateral deformation (minimum margin of 1.33) before failure of primary elements of the lateral-force-resisting system and significant potential for instability or collapse would be expected. As indicated in the *Commentary* to the *NEHRP Recommended Provisions* (BSSC, 1997), significantly better performance is expected of new structures when subjected to their design earthquake ground motions. Such structures are anticipated to provide a margin of at least 1.5 against collapse at the design earthquake level. Lower margins were specifically selected for the Life Safety Performance Level under the *Guidelines* to be consistent with historic practice that has accepted higher levels of risk for existing structures, based largely on economic considerations.

It should be noted that for given buildings the relative horizontal and vertical scales shown on this plot may vary significantly, and the margin of deformation between individual performance levels may not be as large as indicated in this figure. Figure C2-4 is a similar curve, representative of the behavior of a nonductile, or brittle, structure. Note that for such a structure, there may be relatively little margin in the response that respectively defines the three performance levels.

For a given structure and design earthquake, it is possible to estimate the overall deformation and force demand on the structure and, therefore, the point on the corresponding curves shown in Figures C2-3 or C2-4 to which the earthquake will push the building. This either will or will not correspond to the desired level of performance for the structure. When structural/seismic rehabilitation is performed, modifications to the structure are made to alter its strength, stiffness, or ability to dampen or resist induced deformations. These actions will alter the characteristics of both the shape of the curves in these figures and the deformation demand produced by the design earthquake on the building, such that the expected performance at the estimated deformation level for the rehabilitated structure is acceptable.

In addition to the three performance levels, two performance ranges are defined in the *Guidelines* to allow users greater flexibility in selecting design Rehabilitation Objectives. Specific design parameters for use in designing within these ranges are not provided. The Damage Control Performance Range represents all those behavior states that occur at lower

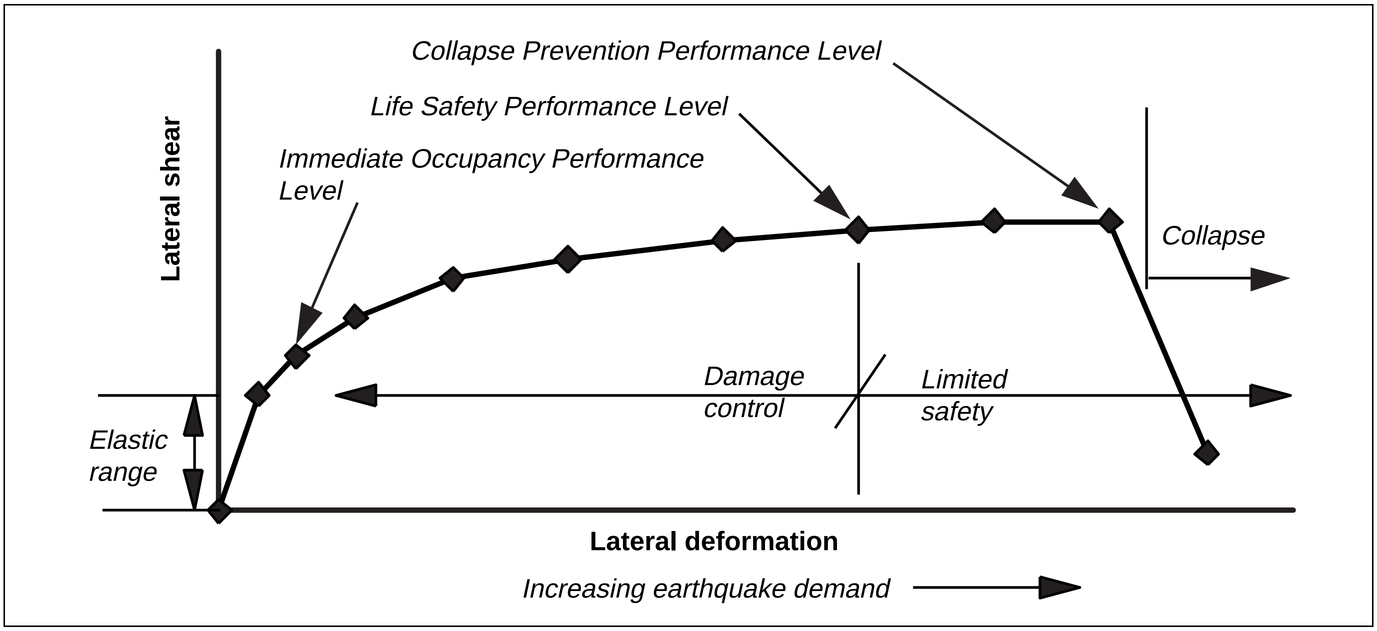


Figure C2-3 Performance and Structural Deformation Demand for Ductile Structures

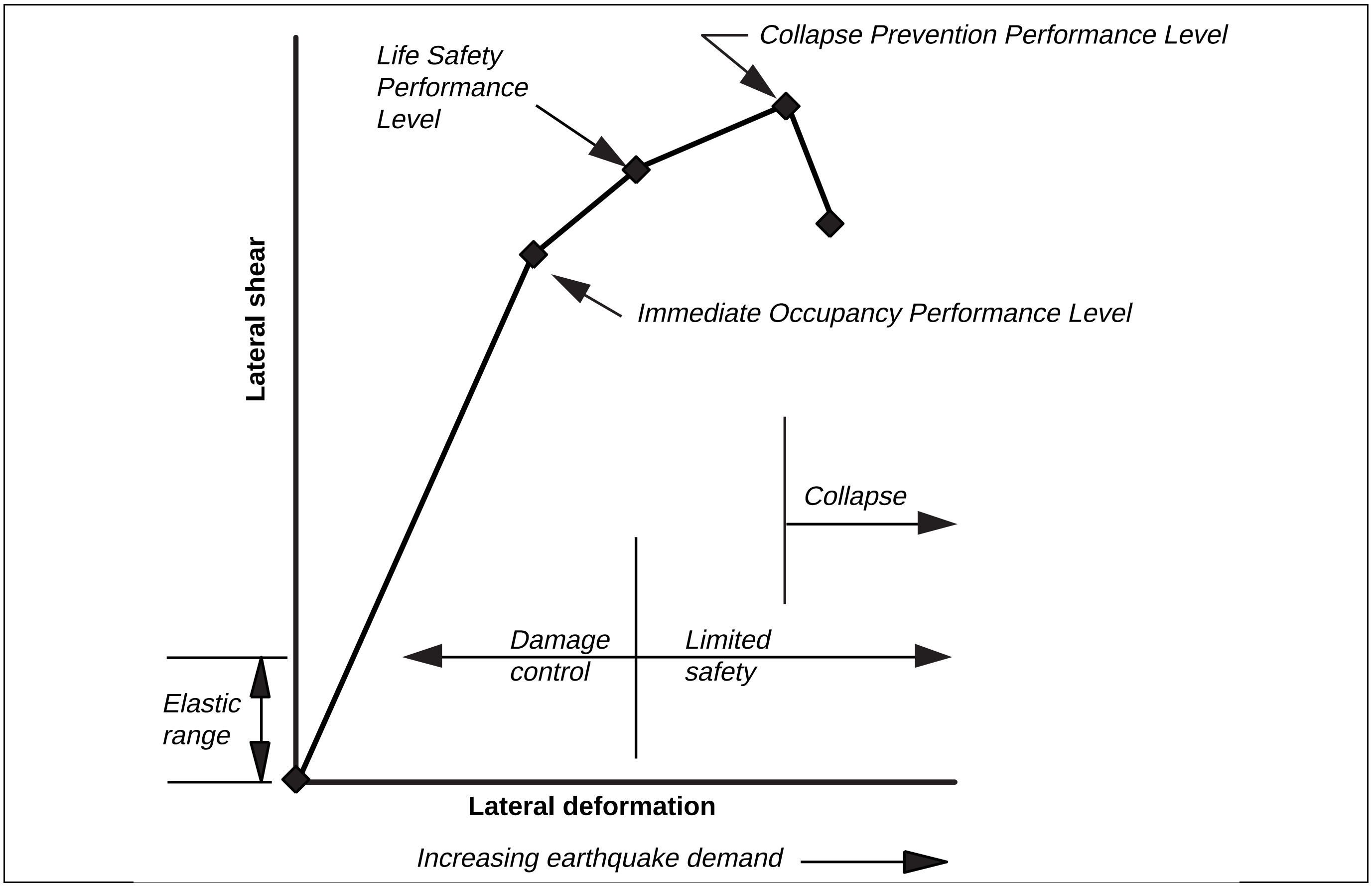


Figure C2-4 Performance and Structural Deformation Demand for Nonductile Structures

levels of lateral deformation than that defined for Life Safety. At the lower levels of deformation contained within this range, the structure would behave in a predominantly elastic manner. At upper levels of deformation within this range, the structure may experience significant inelastic behavior. In general, the more inelastic behavior the structure experiences, the greater the extent of structural damage expected.

The Limited Safety Performance Range of behavior includes all those behavior states that occur at lateral deformation levels in excess of the Life Safety Performance Level, including, possibly, collapse states. Designing for performance within the Limited Safety Range may imply a significant risk of life and economic loss.

C2.5.2 Nonstructural Performance Levels

Nonstructural Performance Levels define the extent of damage to the various nonstructural components included in a building, such as electrical, mechanical, plumbing, and fire protection systems; cladding, ceilings, and partitions; elevators, lighting, and egress; and various items of tenant contents such as furnishings, computer systems, and manufacturing equipment. Although structural engineers typically have relatively little input to the design of these items, the way in which they perform in an earthquake can significantly affect the operability and even fitness for occupancy of a building following an earthquake. Even if a building's structure is relatively undamaged, extensive damage to lights, elevators, and plumbing and fire protection equipment could render a building unfit for occupancy.

There are three basic issues related to the performance of nonstructural components. These are:

- Security of component attachment to the structure and adequacy to prevent sliding, overturning, or dislodging from the normal installed position
- Ability of the component to withstand earthquake-induced building deformations without experiencing structural damage or mechanical or electrical fault
- Ability of the component to withstand earthquake-induced shaking without experiencing structural damage or mechanical or electrical fault

Until recently, the building codes for new construction were generally silent on the issue of how to design nonstructural components for seismic performance. Even in contemporary codes, the consideration of nonstructural performance is generally limited to the security of attachment of components to the structure, specifically with regard to the protection of occupant life safety. Consequently, widespread vulnerabilities of nonstructural components exist within the building inventory.

Mitigation of nonstructural seismic vulnerabilities is a complex issue. Many nonstructural components, if adequately secured to the structure, are seismically rugged. Further, retroactive provision of appropriate anchorage or bracing for some nonstructural components can be implemented very economically and without significant disruption of building function. However, mitigation of some vulnerabilities, such as provision of bracing for mechanical and electrical components within suspended ceiling systems, or the improvement of the ceiling systems themselves, can result in extensive disruption of occupancy and can also be quite costly.

C2.5.2.1 Operational Nonstructural Performance Level (N-A)

In designing for the Operational Nonstructural Performance Level, it will typically be necessary to secure all significant nonstructural components. Further, it will also be necessary to ensure that the components required for normal operation of the facility can function after being subjected to the displacements and forces transmitted by the structure. In order to obtain such assurance, it may be necessary to conduct tests of the behavior of prototype components on shaking tables, using motion that simulates that which would be transmitted to the component by the building structure. This is a tedious and extremely costly process that is beyond the economic capabilities of most owners. However, the nuclear industry has typically incorporated such procedures in the design of critical safety systems for their facilities.

C2.5.2.2 Immediate Occupancy Nonstructural Performance Level (N-B)

It will generally be more practical for most owners to design for the Immediate Occupancy Nonstructural Performance Level. At this level, all major nonstructural components are secured and prevented from sliding, toppling, or dislodging from their mountings. Since many nonstructural components are structurally rugged, it would be expected that most would be in an operable condition, assuming that the necessary power and other utilities are available. However, even attaining this level of nonstructural performance can be quite costly, as it may require modification of the installation of systems such as piping, ductwork, and ceilings throughout the building.

C2.5.2.3 Life Safety Nonstructural Performance Level (N-C)

The Life Safety Nonstructural Performance Level is obtained by structurally securing those nonstructural components that could pose a significant threat to life safety if they were to be dislodged by earthquake shaking. The primary difference between this level and that for Immediate Occupancy is that many small, lightweight components that are addressed under the Immediate Occupancy Performance Level are deemed not to be a significant life hazard and are not addressed under the Life Safety Performance Level. In addition, the Immediate Occupancy Performance Level requires somewhat more control of building lateral deflections than does the Life Safety Performance Level, in order to control to a somewhat greater degree the extent of damage resulting from in-plane deformation of elements such as cladding and partitions.

C2.5.2.4 Hazards Reduced Nonstructural Performance Level (N-D)

The Hazards Reduced Nonstructural Performance Level is similar to the Life Safety Performance Level except that the components that must be secured are limited to those that, if dislodged, would pose a major threat to life safety, capable of severely injuring a number of people. This would include elements such as parapets and exterior cladding panels. However, components such as individual light fixtures or HVAC ducts would not be addressed, nor would building deflections be limited as a method of controlling damage to items such as partitions and doors. This Performance Level provides for cost-effective mitigation of the most serious nonstructural hazards to life safety.

C2.5.2.5 Nonstructural Performance Not Considered (N-E)

No commentary is provided for this section.

C2.5.3 Building Performance Levels

No commentary is provided for this section.

C2.6 Seismic Hazard

Until the publication of ATC-3-06 (1978), the consideration of seismic hazards by the building codes was performed in a highly qualitative manner. The codes contained seismic hazard maps that divided the nation into a series of zones of equivalent seismicity. Until the mid-1970s, these maps contained four zones: (0) negligible seismicity, (1) low seismicity, (2) moderate seismicity, and (3) high seismicity. In the mid-1970s, zone 3 was further divided to produce another zone, zone 4, encompassing regions within 20 miles of major active faults. The classification of sites within the various zones was based on the historic seismicity of the region. If there were no historic reports of damaging earthquakes in a region it was classified as zone 0. If there were many large damaging earthquakes in an area, it was classified as zone 3, or later zone 4. Design force levels for structures were directly tied to the seismic zone in which a building was sited; however, these force levels were not correlated in any direct manner with specific ground motion spectra.

The ATC (1978) publication introduced the concept of acceleration response spectra into the design process and suggested that the design force levels then being used for design in the zones of highest seismicity corresponded to design response spectra that had an effective peak ground acceleration of 0.4g. This publication further suggested that this level of ground motion roughly corresponded with that which would be exceeded roughly one time every 500 years, having approximately a 10% probability of exceedance in 50 years. In place of seismic zones, hazard maps published with the ATC document represented seismic hazard in terms of two ground motion parameters, A_a and A_v, plotted by county on the maps. The A_a parameter represented an effective peak ground acceleration—that is, the acceleration that a perfectly rigid structure, having a period of 0 seconds, would effectively experience if subjected to the ground motion. The A_v parameter represented the response acceleration corresponding to the effective peak response velocity that a structure would experience when subjected to this ground motion. While neither the ATC document itself nor the maps published with the document were immediately adopted into the building codes, it became accepted doctrine that the design forces specified in the building codes, still based on the old seismic zonation maps, represented hazards with a 10%/50 year exceedance probability, and that the design procedures contained in the building codes provided a performance level for this ground shaking that would ensure protection of the life safety of building occupants as well as control damage in most structures to levels that would be repairable under these levels of ground shaking. Further, it was considered by many of the participants in the ATC project that structures designed for values of A_a and A_v equal to 0.4g, together with the detailing requirements recommended in the document for that level of design, would be able to survive any earthquake of the type likely to be experienced in California. Together, these combined performance levels were considered to provide a socially acceptable level of risk.

During the 1980s and 1990s, seismologists' ability to estimate ground shaking hazard levels improved significantly. This was largely due to the occurrence of a number of moderate- to large-magnitude earthquakes in regions of California in which there were many strong motion instruments. This provided a wealth of data on the variation of ground motion correlated with distance from the causative fault, magnitude, site characteristics, and other parameters. At the same time, the use of paleoseismic techniques permitted re-evaluation of the recurrence rates of rare, large-magnitude earthquakes in areas such as the New Madrid region in the Mississippi embayment, the region around Charleston, South Carolina, and the Pacific Northwest. Based on this re-evaluation, several inconsistencies in the previous definition of acceptable risk, as described above, became apparent. First, it appeared clear that the 0.4g effective peak ground acceleration, previously assumed to be representative of ground motion with a 10%/50 year exceedance level in zones of high seismicity, significantly underestimated the motion that would be experienced in the near field of major active faults. Also, it became apparent that in areas that experienced truly infrequent, but very large-magnitude earthquakes, such as the Mississippi embayment, structures designed to the 10%/50 year hazard level might not have adequate seismic resistance to resist even historic earthquakes without collapse.

In response, the 1988 *NEHRP Recommended Provisions for New Buildings* published a second series of seismic risk maps, providing A_a and A_v contours for 2% probability of exceedance in 50 years (termed a 2%/50 year exceedance level in the *Guidelines*) in addition to the standard 10%/50 year maps published with previous editions. However, there was no consensus that it was appropriate to actually design buildings for these levels of ground motion. The design community was divided on this issue, some believing that the 10%/50 year maps did not provide adequate protection of the public safety, and others believing that design for the 2%/50 year hazards would be economically impractical.

In the early 1990s, the United States Geological Survey (USGS) developed a new series of ground motion hazard maps, utilizing the latest seismological knowledge. The BSSC attempted to incorporate these maps for use in the 1994 *NEHRP Recommended Provisions*; however, the necessary consensus was not achieved. Some engineers in the western United States believed that the hazards represented by the proposed 10%/50 year maps provided values that were unacceptably high for design purposes in the regions surrounding major active faults, and unacceptably low for design purposes in regions remotely located from such faults. Further, it was felt by some that these maps still did not adequately address the possibility of infrequent, large-magnitude earthquakes in the eastern United States.

The *NEHRP Recommended Provisions* (BSSC, 1997) update process included the formation of a special Seismic Design Procedures Group (SDPG), consisting of earth scientists from the USGS and engineers engaged in the update process. The SPDG was charged with the responsibility of working with the USGS to produce ground motion maps incorporating the latest earth science procedures, and with appropriate design procedures to allow use of these maps in the *Recommended Provisions*. The SDPG determined that rather than designing for a nationwide uniform hazard—such as a 10%/50 year or 2%/50 year hazard—it made more sense to design for a uniform margin of failure against a somewhat arbitrarily selected maximum earthquake level.

This maximum earthquake level was termed a Maximum Considered Earthquake (MCE) in recognition of the fact that this was not the most severe earthquake hazard level that could ever affect a site, but it was the most severe level that it was practical to consider for design purposes. The SDPG decided to adopt a 2%/50 year exceedance level definition for the MCE in most regions of the nation, as it was felt that this would capture recurrence of all of the large-magnitude earthquakes that had occurred in historic times.

There was concern, however, that the levels of ground shaking derived for this exceedance level were not appropriate in zones near major active faults. There were several reasons for this. First, the predicted ground motions in these regions were much larger than those that had commonly been recorded by near field instrumentation in recent magnitude 6 or 7 California events. Second, it was noted, based on the observed performance of buildings in these earthquakes, that structures designed to the code had substantial margin against collapse for ground shaking that is much larger than that for which the building had nominally been designed; in the judgment of the SDPG members, this margin represented a factor of at least 1.5. Consequently, it was decided to adopt a definition of the MCE in zones near major active faults that consisted of the smaller of the probabilistically estimated 2%/50 year motion or 150% of the mean ground motion calculated for a deterministic characteristic earthquake on these major active faults, and to design all buildings, regardless of location, to provide for protection of occupant life safety at earthquake ground shaking levels that are 1/1.5 times (2/3) of the MCE ground motion.

Except in zones near faults with very low recurrence rates, deterministic estimates of ground motion typically result in smaller accelerations than do the probabilistic 2%/50 year estimates of ground motion. The SDPG considered it inappropriate to permit design of structures for lower levels of ground motion than that required by the 1994 *NEHRP Recommended Provisions*, in zones of high seismicity. Consequently, the definition of the MCE incorporated a transition zone between those regions where the MCE has a probabilistic definition and those where there is a deterministic definition; that is, in which the ground motion is taken at 150% of the levels required by the 1994 *NEHRP Provisions*.

The implied performance of buildings designed to the 1997 *NEHRP Recommended Provisions*, assuming the SDPG recommendations are ratified, is related to, but somewhat different from, that which historically has been defined as being an acceptable risk. Specifically, it is implied that buildings conforming to the 1997 *Recommended Provisions* would be able to withstand MCE ground shaking without collapse, and withstand design level ground shaking (2/3 of MCE) at reduced levels of damage associated with both protection of occupant safety and provision of reasonable assurance that the building could be repaired and restored to service.

The calculations of probabilistic ground motions conducted by the USGS as a basis for the response acceleration maps incorporated a number of parameters with significant uncertainties. Potential variation and uncertainty in the values of the most significant parameters, such as the probability of events of varying magnitudes and rupture mechanisms occurring along a given source and the variability of attenuation of ground motion over distance, were considered directly in the probabilistic calculations. Uncertainties in many other parameters were not directly accounted for. Initial studies conducted by the USGS of the potential effects of these other uncertainties indicate that the mapped values represent estimates for which there is a high degree of confidence (about the mean plus one standard deviation level) of non-exceedance at a given probabilistic level.

The *Guidelines* have adopted the same definition of the MCE proposed for adoption in the 1997 *NEHRP Recommended Provisions*, as described above, and have designated it Basic Safety Earthquake 2 (BSE-2). However, the *Guidelines* have not directly adopted the concept of a design earthquake, at 2/3 of the MCE level, as proposed for the *Recommended Provisions*. This was not adopted because this design earthquake would have a different probability of exceedance throughout the nation, depending on the seismicity of the particular region. It was felt such an event would be inconsistent with the intent of the *Guidelines* to permit design for specific levels of performance for hazards that have specific probabilities of exceedance selected by the design team. Consequently, instead of adopting the design earthquake concept, it was decided to adopt the Basic Safety Earthquake 1 (BSE-1).

The BSE-1 is typically taken as that ground motion with a 10%/50 year exceedance probability, except that it need never be taken as larger than 2/3 of the BSE-2 ground motion. The 10%/50 year exceedance probability is consistent with that level of hazard that has traditionally been assumed to be an acceptable basis for design in the building codes for new construction. The limitation of 2/3 of the MCE ground motion was adopted so that design requirements for the BSO, defined in Section 2.4.1, would not be more severe than the design requirements for new construction under the 1997 *NEHRP Provisions*.

Ground shaking hazards may be determined by either of two procedures. Section 2.6.1 of the *Guidelines* provides a general procedure in which spectral response acceleration parameters are obtained by reference to the maps in the package distributed with the *Guidelines*. These parameters are then adjusted, if required, to the desired exceedance probability, and modified for site class effects. The resulting parameters are sufficient to allow development of a complete acceleration response spectrum that is directly referenced by the analysis procedures of Chapters 3 and 9. Section 2.6.2 provides general guidance for the application of site-specific procedures in which regional seismicity and geology and individual site characteristics are considered in the development of response spectra.

On a regional basis, the maps referenced in the general procedure may provide reasonable estimates of the response accelerations for the indicated hazard levels. However, these estimates may be insufficiently conservative for some sites, including those with particularly soft soil profiles or soils subject to seismic-induced instability, and sites located in the near field of a fault. Since many of the structural provisions of the *Guidelines* incorporate lower margins of safety than do the FEMA 222A (BSSC, 1995) provisions, it is important that ground motion characterizations used as the basis of design not be underestimated. Use of the Site-Specific Procedures for these sites will generally result in improved estimates of the likely ground shaking levels, and increase design reliability. Use of the Site-Specific Procedures is also recommended for buildings with Enhanced Rehabilitation Objectives, because such objectives are typically adopted for important buildings in which the greater design reliability provided by a site-specific hazard estimate is appropriate. Site-specific procedures should also be used when a Time-History Analysis is to be performed as part of the rehabilitation procedure, since the development of site-specific ground motions is commensurate with the greater effort required for the structural analysis, and the greater expectations for reliability common to buildings analyzed by that technique.

C2.6.1 General Ground Shaking Hazard Procedure

In the general procedures, reference is made to a series of hazard maps to obtain key spectral response acceleration parameters. These acceleration parameters, when adjusted for probability of exceedance and for site class effects, are sufficient to define an acceleration response spectrum suitable for use for analysis and design. Two sets of two maps are in the map package distributed with the *Guidelines*. One set of maps provides contours of the key response acceleration parameters for the MCE hazard level, as defined in Section 2.4. These maps were developed by the USGS for inclusion in the 1997 *NEHRP Recommended Provisions* in a joint project with the BSSC, known as Project ’97, and incorporate the latest scientific thought on ground motion estimation as of early 1996. The second set of maps was also developed by the USGS as part of the same project, using a 10%/50 year exceedance probability. Other ground shaking demand maps can be used, provided that 5%-damped response spectra are developed that represent the ground shaking for the desired earthquake return period, and that the site soil classification is considered.

For each hazard level, the maps provide contours of the parameters S_S and S_1. The S_S parameter is the 5%-damped, elastic spectral response acceleration for rock sites (class B) at a period of 0.2 seconds. The S_1 parameter represents the 5%-damped, elastic spectral response acceleration for rock (class B) sites at a period of 1.0 second. In the period range of importance to the response of most structures, acceleration response spectra can be represented by a bilinear curve, consisting of a constant response acceleration at short periods and a constant response velocity at longer periods. Since spectral response acceleration is related to pseudo-spectral response velocity by the equation:

$$S_a = \omega S_v = \frac{2\pi}{T} S_v \qquad \text{(C2-1)}$$

where S_a is the spectral acceleration, ω is the radial frequency of periodic motion, T is the period of motion, and S_v is the pseudo-spectral velocity, then, in the constant velocity range of response, spectral acceleration at any period can be related to that at a one-second period by the factor $1/T$. Thus, the two spectral response acceleration parameters, S_S and S_1, when adjusted for exceedance probability and site class, completely define a response spectrum curve useful for design purposes.

C2.6.1.1 Mapped MCE Response Acceleration Parameters

The MCE maps in the package distributed with the *Guidelines* are the same as those developed by the

SDPG for use in the 1997 *NEHRP Recommended Provisions*. As proposed for use there, the spectral values obtained from these maps would be reduced by a factor of 2/3 to arrive at design spectral values. The *Recommended Provisions* would then provide criteria for design to a performance level within the Damage Control Performance Range, having somewhat more margin against failure (estimated at 150%) than the Life Safety Performance Level, defined in the *Guidelines* with a margin of 133% against failure. In the *Guidelines*, the BSE-2 response accelerations are used to evaluate the ability of structures to meet the Collapse Prevention Performance Level, when designing to achieve the BSO.

In developing acceptance criteria for component actions, the following criteria are set. The permitted inelastic deformation demand for a primary element is set at 75% of the deformation level at which significant strength loss occurs. Although most structures have sufficient redundancy so that collapse would not occur at the loss of the first primary element, this would imply a minimum margin against failure for the Life Safety Performance Level of 1/0.75 or 1.33, apart from inaccuracies inherent in the analysis method. In a similar, but far less rigorous manner, the SDPG, the committee responsible for development of the new NEHRP maps and the corresponding design procedure, judged that the minimum margin against failure contained in the *NEHRP Provisions* is 150%. This was not based on any evaluation of actual acceptance criteria contained in the *NEHRP Provisions*, but rather the judgment that appropriately constructed buildings designed to NEHRP Seismic Performance Category D (or Zone 4 of the 1994 UBC) criteria should not encounter serious problems until ground motion levels of at least 0.6g. The ratio of 0.6g (the judgmentally selected minimum limiting ground motion) to the contemporary design value of 0.4g (for A_a and A_v or Z in the 1994 UBC) resulted in the projected margin of 150%. This 150% is directly related to the 2/3 reduction between the MCE and Design Based Earthquake (DBE) maps in the *NEHRP Provisions* (2/3 = 1/1.5).

It is important to note that the BSE-2 hazards defined by these maps cannot be associated with a particular exceedance probability. Although the hazards indicated for most regions covered by the map have been probabilistically calculated as having a 2%/50 year exceedance probability, the regions surrounding major active fault systems, such as those in coastal California, have been adjusted to include deterministic estimates of ground shaking for specific maximum earthquake events on each of the several faults known to be present in the region. Consequently, the values of the spectral response accelerations obtained from these maps should not be used when attempting to develop hazards with a particular exceedance probability, in accordance with Section 2.6.1.3.

C2.6.1.2 Mapped 10%/50 Year and BSE-1 Response Acceleration Parameters

The probabilistic maps in the package distributed with the *Guidelines* provide contours for the spectral response acceleration parameters at a uniform 10%/50 year exceedance probability. These acceleration parameters, once adjusted for site class effects and to limit maximum accelerations to 2/3 of those of BSE-2, can be used directly to evaluate the ability of structures to meet the Life Safety Performance Level when designing to achieve the BSO. In addition, these acceleration parameters, having a uniform exceedance probability, can be used to derive response acceleration parameters for any exceedance probability, using the procedure of Section 2.6.1.3.

C2.6.1.3 Adjustment of Mapped Response Acceleration Parameters for Probability of Exceedance

An examination was performed of typical hazard curves used by the USGS to construct the ground motion maps distributed with the *Guidelines*. A log-log plot of these curves in a domain of annual frequency of exceedance (or return period) versus response spectral acceleration is nearly linear between probability of exceedance levels of 2% and 10% in 50 years. Therefore, for regions in which the BSE-2 maps directly provide spectral response acceleration parameters with a 2%/50 year exceedance rate, a linear interpolation on a log-log plot of spectral response acceleration versus return period can be made to find the response spectral accelerations for any desired probability levels within these ranges. This approach is applicable anywhere that the short period response acceleration parameter, S_S, is less than 1.5g. Equation 2-1 provides a closed form solution for this logarithmic interpolation. Equation 2-2 allows return period, P_R, to be determined for any defined probability of exceedance in 50 years.

In regions where the short period spectral response accelerations provided on the BSE-2 map are equal to or greater than 1.5g, the response acceleration contours on the maps are based on deterministic rather than probabilistic concepts. In these regions the BSE-2 map values cannot be used to interpolate for intermediate exceedance rates. Instead, Equation 2-3 is used to estimate the spectral response acceleration parameters at arbitrary return periods by extrapolating from the 10%/50 year value, obtained from the maps with an approximate hazard curve slope, represented by the coefficient *n*. These approximate hazard curve slopes have been estimated on a regional basis. They were derived by examining the typical hazard curves developed by the USGS for representative sites in each of the major seismicity zones including California, the Pacific Northwest, the Intermountain region, Central, and Eastern United States and taking an approximate mean value for these sites. A similar approach is used to estimate spectral response accelerations parameters for hazards with exceedance rates greater than 10%/50 years in all regions of the nation, as the logarithmic extrapolation that may be used between exceedance rates of 2%/50 years and 10%/50 years is not valid outside this range.

C2.6.1.4 Adjustment for Site Class

The definitions of the site classes, A through F, and site coefficients, F_a and F_v, were originated at a workshop on site response held at the University of Southern California in November 1992. In that workshop, convened by the National Center for Earthquake Engineering Research (NCEER), Structural Engineers Association of California (SEAOC), and BSSC (Martin and Dobry, 1994; Rinne, 1994), consensus values for the ratios of response spectra on defined soil profile types relative to rock for the short-period range and long-period range were developed on the basis of examination of empirical data on site amplification effects (especially data from the 1989 Loma Prieta earthquake) and analytical studies (site response analyses). The response spectral ratios relative to rock (site class B) were designated F_a for the short-period range (nominally at a period of 0.3 second) and F_v for the long-period range (nominally at a period of 1.0 second). The recommendations of this workshop for both the soil profile types and the site factors F_a and F_v were adopted by the BSSC for the 1994 edition of *NEHRP Recommended Provisions* (BSSC, 1995).

The 1994 *NEHRP Recommended Provisions* defined values of F_a and F_v in Tables 2-13 and 2-14 for ground motions with effective peak ground accelerations on rock sites equal to or less than 0.40g—the highest value used in the *Provisions*. For effective peak ground accelerations on rock equal to 0.50g, the values of F_a and F_v were similarly obtained by using the values recommended by the workshop. The workshop did not present recommendations for values of F_a and F_v for effective peak ground accelerations on rock greater than 0.50g. In fact, because of a lack of recorded data on site amplification effects at higher acceleration levels, there is increasing uncertainty as to appropriate values of F_a and F_v for higher accelerations. It is not clear that the site factors would continue the trend of reduction with increasing acceleration. Therefore, values of F_a and F_v for effective peak ground accelerations on rock exceeding 0.50g have been obtained using the values of F_a and F_v defined by the workshop for an acceleration coefficient of 0.50. Consistent with the workshop recommendations, site-specific studies incorporating dynamic site response analyses are recommended for soft soils (profile E) for effective peak ground accelerations on rock equal to or greater than 0.50g. Therefore, values of F_a and F_v are not presented in Tables 2-13 and 2-14 for Type E soils for effective peak ground accelerations on rock equal to or greater than 0.50g.

It should be noted that, in contrast to the site factors in previous editions of the *NEHRP Recommended Provisions for New Buildings* and in the *Uniform Building Code* (ICBO, 1994), the new site factors incorporate two significant features. First, there are factors for short periods as well as long periods, whereas the previous site factors were only for long periods. This reflects the empirical observation (especially from the 1989 Loma Prieta earthquake) that short-period as well as long-period ground motions are amplified on soil relative to rock, especially for lower acceleration levels. Second, the factors are a function of acceleration level, whereas the previous factors were independent of the acceleration. This reflects the nonlinearity of soil response; soil amplifications decrease with increasing acceleration due to increased damping in the soil. In common with the previous site factors, the new site factors increase as the soils become softer, but the new factors are higher than the previous factors at the lower acceleration levels.

C2.6.1.5 General Response Spectrum

Section 2.6.1.5 provides guidelines for the development of a general acceleration response spectrum based on the values of the design response acceleration parameters, S_{XS} and S_{X1}, that include necessary adjustments for probability of exceedance and site class effects. The shape of this general response spectrum incorporates two basic regimes of behavior—a constant response acceleration range at short periods and a constant response velocity range at long periods, in which, as previously described, response acceleration varies inversely with structural period. The transition between the two regimes occurs simply at that period where acceleration values calculated assuming constant response velocity would exceed those of the constant acceleration regime.

This general spectrum is a somewhat simplified version of the spectrum presented by Newmark and Hall (1982). The Newmark and Hall spectrum, derived from a statistical evaluation of a number of historic earthquake ground motion recordings, actually included four distinct domains. In addition to the constant response acceleration and constant response velocity domains included in the spectra contained in the *Guidelines*, the Newmark and Hall spectrum included a constant response displacement domain at very long periods, in which response acceleration varies with the inverse of the square of structural period ($1/T^2$), and a transition zone in the very short period range, in which the response acceleration increased rapidly from the effective peak ground acceleration for infinitely rigid structures (natural period of 0 seconds) to the constant response acceleration value.

The simplified version of the general spectrum presented in the *Guidelines* is sufficiently accurate for use for most structures on most sites, and adequately represents the response of structures to the random vibratory ground motions that dominate structural response on sites located 10 or more kilometers from the fault rupture surface. However, it does potentially overstate the response acceleration demand for very rigid (short-period) structures and for very flexible (long-period) structures. In addition, it potentially understates the effects of the impulsive-type motions that have been experienced on sites located within a few kilometers of the fault rupture surface. These impulsive motions can cause very large response in structures with periods ranging from perhaps one second to as long as four seconds. For buildings within this period range, and located on sites where such impulsive motions are likely to be experienced, the site-specific procedures should be considered.

The approach adopted by the *Guidelines* for construction of a general response spectrum is similar to the approach that has been adopted by the *NEHRP Recommended Provisions* for designs based on the equivalent lateral force technique. In the development of the *Guidelines*, it was decided, for several reasons, to neglect the very short period range of the spectrum, in which response accelerations are somewhat lower than those in the constant acceleration domain. First, it was the feeling of the development team that very few building structures actually have effective periods within this very short period range, especially when the likely effects of soil structure interaction and degradation due to inelastic behavior are considered. Second, designing for acceleration response within this very short period range could lead to unconservative designs. This is because as a structure responds inelastically to earthquake ground motion, its stiffness will tend to degrade somewhat, resulting in a longer effective period. Therefore, if a structure has a very short period and is designed for the resulting reduced accelerations, under the effects of stiffness degradation it could shift to a somewhat longer period and experience more acceleration response than that for which it had been designed.

The decision to neglect the constant displacement domain of the spectrum was made for several reasons. First, at the time of the *Guidelines* development, there were no readily available rules for determining the period at which the constant displacement domain initiates. This transition period would appear to be a function of the site class, as well as the location and position of the individual site with respect to the fault rupture plane and direction of rupture propagation. Such effects are very difficult to incorporate in a series of general purpose rules. The *NEHRP Recommended Provisions* have adopted a period of four seconds as a general guideline for this transition period, when performing dynamic analyses. However, this period is somewhat arbitrary and may produce unconservative designs on some sites. Second, relatively few structures that will be rehabilitated using the *Guidelines* are likely to have periods long enough to fall within this domain. Those structures that do have such long periods are likely to be quite tall and, therefore, of the class for which site-specific ground motion determination is recommended. Nothing in these *Guidelines* would prevent the adoption of spectra with a constant displacement domain if it is developed on the basis of site-specific study by a knowledgeable earth scientist or geotechnical engineer.

It should be noted that spectra generated using site-specific procedures may not have well-defined constant acceleration, constant velocity, and constant displacement domains, although they will typically resemble spectra that have these characteristics. For such spectra, it is recommended that, at least for consideration of first mode response, the effective value of the response acceleration for very short periods be taken as not less than that obtained at a period of 0.3 seconds, or that which would be derived by the general procedure. Consideration could be given to using the value of accelerations for very short period response when evaluating the effect of higher modes of response.

The general response spectrum has been developed for the case of 5%-damped response. A procedure is also provided in the *Guidelines* for modifying this 5%-damped spectrum for other effective damping ratios. These modification factors are based on the recommendations contained in Newmark and Hall (1982) for median estimates of response, except that for damping ratios *ß* of 30% and greater, more conservative estimates have intentionally been used, consistent with the approach adopted for seismic-isolated structures in the 1994 *NEHRP Provisions*. Again, it is important to note that structures may not respond with the same effective damping when they are subjected to impulsive-type motions, as they do when subjected to the more typical random vibratory motions represented by the general response spectrum.

C2.6.2 Site-Specific Ground Shaking Hazard

In developing site-specific ground motions, both response-spectra, and acceleration time histories, it should be kept in mind that the characteristics of the ground motion may be significantly influenced by not only the soil conditions but also the tectonic environment of the site. Of particular importance for long-period structures is the tendency for near-source ground motions to exhibit a long-period pulse (e.g., Sommerville and Graves, 1993; Sadigh et al., 1993; Boatwright, 1994; Heaton and Hartzell, 1994; Heaton et al., 1995). The existence of very hard rock in the eastern U.S. (relative to typical rock in the western U.S.) results in an increase in the high-frequency content of ground motion in the east as compared to that in the west (e.g., Boore and Joyner, 1994). Duration of strong ground shaking is closely related to earthquake magnitude and also dependent on distance and site conditions (e.g., Dobry et al., 1978).

A greater number of acceleration time histories is required for nonlinear procedures than for linear procedures because nonlinear structural response is much more sensitive than linear response to characteristics of the ground motions, in addition to the characteristics of response spectral content. Thus, nonlinear response may be importantly influenced by duration as well as by the phasing and pulse sequencing characteristics of the ground motions.

C2.6.3 Seismicity Zones

No commentary is provided for this section.

C2.6.4 Other Seismic Hazards

No commentary is provided for this section.

C2.7 As-Built Information

Prior to evaluating an existing building and developing a rehabilitation scheme, as much existing data as are available should be gathered. This includes performing a site visit, contacting the applicable building department that may have original and modified plans and other documents, and conducting meetings with the building owners, managers, and maintenance engineers who may have direct knowledge of the condition and construction of the building and its past history, as well as files and documents with similar valuable information. Also, if the original design professionals (e.g., architects and engineers) and construction contractors and subcontractors can be identified, additional information—such as design bases, calculations, change orders, shop drawings, and test reports—may be attainable. After available documents are reviewed, field surveys should be made to verify the accuracy and applicability of the available documents. When documents are not available, field measurements are required. A program for destructive and nondestructive tests should be developed and implemented.

The importance of attempting to obtain all available documentation of a building's construction prior to proceeding with an evaluation and rehabilitation program cannot be overemphasized. Without a clear understanding of the construction of a building, it is difficult to predict its response to future seismic demands and, therefore, to determine an appropriate program for rehabilitation. If documentation of the building's construction is not available, it is often necessary to conduct extensive surveys of the building to allow development of this documentation. In most buildings, critical details of the structural system are obscured from view by architectural finish, fireproofing, and the structural elements themselves. Therefore, destructive examination may often be required to obtain an appropriate level of information.

For those buildings for which good documentation, in the form of original design drawings and specifications, is available, it should not be assumed that these documents represent the actual as-built or current configuration of the structure. As a minimum, a general survey of the structure should be conducted to confirm that the construction generally conforms to the intent of the documents and that major modifications have not been made. It may also be advisable to confirm that certain critical details of construction were actually constructed as indicated.

Though some useful information, such as probable material strengths, can be obtained by reference to the building codes and standard specifications commonly in use at the time of construction, such data should be used with caution. Since many municipalities are slow in their adoption of current standards, buildings constructed in one era may actually have been designed in accordance with earlier standards. Also, there is no guarantee that a building has actually been designed and constructed in conformance with the applicable code requirements.

C2.7.1 Building Configuration

Most buildings have a substantial lateral-load-resisting system, although this may not be adequate to achieve the Rehabilitation Objectives. Often, a significant portion of a building's resistance to lateral demands will be provided by elements that were not specifically intended by the original designer to serve this purpose. In particular, the walls of many buildings, although not intended to participate in lateral force resistance, will in actuality do so, and may not only provide substantial resistance but also alter the manner in which the primary system behaves. These elements can also introduce critical irregularities into a building's lateral-load-resisting system. Architectural walls and partitions can affect the stiffness of structural elements and also introduce soft story and torsional conditions into otherwise regular buildings. It is important to consider these aspects when developing a concept of the building's configuration.

C2.7.2 Component Properties

In order to define the strength and deformation characteristics of the building and its elements, one must know the relevant properties of the components, including the cross sections present, material strengths, and connectivity details. Since the strength of materials actually present in a structure can vary significantly from that indicated on original construction drawings, testing is the preferred method of ascertaining material strength. In some cases, original construction quality control data—including mill test certificates, concrete cylinder test reports, and similar documentation—may provide a direct indication of the material strengths. Such data should be adequate if the structure has remained in good condition.

It is important to obtain the force-displacement characteristics of the existing elements—whether or not they are to be included in the lateral-force-resisting system—because of the need to determine the deformation compatibility relationships of existing materials with the new materials used in the rehabilitation concepts. When a building responds to ground motion, the demands on nearly all components of the building are altered. There is potential for components that do not provide significant lateral resistance in a structure to experience demands that can result in severe damage. Reinforced concrete buildings with flat slab floors and perimeter shear walls provide a good example. The equivalent frames comprising the flat slabs and columns may provide relatively little lateral-force resistance compared to that of the perimeter shear walls. However, such slabs can be extremely vulnerable to lateral deformations that induce relatively large shear stresses in the column-to-floor-slab connections. Although most engineers would not consider the slabs to be part of the lateral-force-resisting system for such buildings, it is important to quantify the lateral deformation capacity of these components to ensure that earthquake demands are maintained below a level that would result in collapse potential. Therefore, investigation of the properties of such secondary elements may be required.

When determining the deformation capacity of a component, or its ability to deliver load to adjacent components, its strength should be calculated using the expected values of strengths for the materials in the building. The expected strengths are the best estimates of the actual strength of the materials in the building as represented by the average value of strengths that one would obtain from tests on a series of samples. The expected strength is different from the nominal or specified strength that is commonly used when materials are specified for new construction. Typically, the actual strengths of materials in new construction are considerably higher than the specified strengths, which provides an additional margin of safety in new construction. Expected strengths are used in the *Guidelines* for two reasons. First, the use of artificially low values, based on nominal or specified values, would result in poor predictions of building performance. Second, the use of such low values, particularly in nonlinear procedures, could result in underestimation of the actual strength demands on some elements of the system.

In addition to expected strengths, the *Guidelines* require estimates of lower-bound strengths for the evaluation of the adequacy of component force actions during force-controlled behavior.

For many existing buildings, information on the strengths obtained in the original construction is not readily available; hence, it is necessary to determine expected strengths from field or laboratory tests. The individual material sections of the *Guidelines* recommend appropriate types, methods, and numbers of tests to define adequately the material strength of an existing building (see Chapters 5 through 8). Actual strengths of materials within a building may vary from component to component; for example, beams and columns in concrete structures may be constructed of materials having different strengths. Strengths may also be affected by deterioration, corrosion, or both.

The κ factor is used to express the confidence with which the properties of the building components are known, when calculating component capacities. The value of the factor is established from the knowledge that the engineer is able to obtain, based on either access to the original construction documents or surveys and destructive or nondestructive testing of representative components.

Two values for the κ factor have been established, indicating whether the engineer's knowledge of the structure is "minimal" or "comprehensive." Recommendations are given in the material chapters as to the level of investigation required for each class. The numerical values of the κ factor are selected to reward a more detailed investigation of the existing building by requiring the use of a discounted value of the expected capacity to be used for analysis and design purposes when only limited information on the structure is available. When nonlinear procedures are used for a building, a comprehensive level of knowledge should be obtained with regard to component properties; if this were not done, the apparent accuracy of the procedure could be misleading.

Examples of the type of knowledge needed for a reinforced concrete shear wall component, in order to qualify under the two classes of knowledge (κ factors), are as follows:

- "Comprehensive" Class

  a. Original construction documents are available and the construction was subject to adequate inspection. Limited visual access to the building and material testing confirm the provisions of the original documents.

  b. Original construction documents are not available, but full access to critical load path components is available, and an adequate testing and inspection program provides information sufficient to define component properties and to conduct structural analyses. Critical details such as the location and length of reinforcing splices are confirmed.

- "Minimal" Class

  a. Only limited or no construction documentation is available.

  b. Access is provided to some but not all load path elements.

  c. Nondestructive Examination (NDE) provides location of reinforcing bars in the wall and limited exposure provides information on bar size and splice lengths. Limited testing for concrete and steel strengths has been performed, and the strength levels and variation in strength levels are consistent with building construction for the age of the building.

C2.7.3 Site Characterization and Geotechnical Information

Regional geologic maps produced by the USGS, as well as those produced by a number of state and local agencies, can be a good source of basic geotechnical data for a site. Information from the geologic maps could include data relative to the surficial geologic unit mapped in the vicinity of the building site. These maps typically include a brief assessment of engineering parameters and performance characteristics that may be attributed to specific geologic units. Information obtained from topographic maps would be used to evaluate potential effects from landslides occurring either on-site or off-site. Finally, various cities have developed hazard maps that may indicate zones that may be susceptible to landslides, liquefaction, or significant amplification of ground shaking. Information obtained from these sources could be used in assessing the large-scale performance of the site, and the need to obtain site-specific data.

Relevant site information that could be obtained from geotechnical reports would include logs of borings and/or cone penetrometer tests, laboratory tests to determine the strength of the subsurface materials, and engineering assessments that may have been conducted addressing geologic hazards at the site, such as faulting, liquefaction, and landsliding. Information should be obtained from geotechnical reports or other regional studies regarding potential depths of groundwater at the site.

Existing building drawings should be reviewed for relevant foundation data. Information to be derived from these drawings could include:

- Shallow foundations
  - footing elevation
  - permissible bearing capacity
  - size

- Deep foundations
  - type (piles or piers)
  - material
  - tip elevation
  - cap elevation
  - design load

Visual site reconnaissance should be conducted to gather information for several purposes, including confirmation that the actual site conditions agree with information obtained from the building drawings, documentation of off-site development that may have a potential impact on the building, and documentation of the performance of the existing building and adjacent areas to denote signs of poor foundation performance.

C2.7.4 Adjacent Buildings

Although buildings are classically evaluated and designed with the assumption that they are isolated from the influence of adjacent structures, there are many instances in which this is not the case. In older urban centers, many buildings were constructed immediately adjacent to each other, with little if any clearance between the structures. Many such buildings have party walls and share elements of their vertical- and lateral-force-resisting systems. Building adjacency issues may also be important for large complexes of buildings constructed in different phases, over a number of years, and for large buildings provided with expansion joints between portions of the building. It is critical to the rehabilitation process to recognize the potential effects of adjacent structures on building behavior.

In order to evaluate potential building interaction effects, it is necessary to understand the construction and behavior of both buildings. In its simplest form, evaluation requires knowledge as to whether or not adjacent structures actually share elements, such as party walls, and an estimate of how much lateral motion each building is likely to experience so that the likelihood of pounding can be evaluated. This requires that at least a minimum level of information be obtained for the adjacent structure, or structures, as well as the building being rehabilitated. Obtaining as-built information for adjacent structures that have different ownership than the building may be difficult. Most owners will be willing to share available information, although they will be less motivated to do so than the owner for whom rehabilitation work is planned. It will seldom be possible or necessary to obtain material test data for adjacent structures. In many cases, it will be necessary to make informed assumptions as to the adjacent structure's characteristics.

C2.7.4.1 Building Pounding

Building pounding is a phenomenon that occurs when adjacent structures are separated at distances less than the differential lateral displacements that occur in each structure as a result of their earthquake response. As a result, the buildings impact each other, or "pound." Pounding can cause local crushing of the structures, and failure of structural and nonstructural elements located in the zone of impact. In addition, pounding can cause a transfer of kinetic energy and momentum from one structure to another, resulting in significantly different earthquake demands in each structure than would be experienced if pounding did not occur. Key to evaluating the potential effects of impact is identifying whether or not such impacts will occur. Conservatively, if the adjacent structures respond to the earthquake ground motion completely out of phase, impact can occur only if the separation of the adjacent structures is less than the sum of the maximum displacement response of the structures at the level of potential impact. Following this approach, the *Guidelines* suggest that adjacency evaluation should be conducted wherever the adjacent structure is closer to the building than 4% of its height above grade at the location of potential impact. This correlates with the assumption that most structures will not exceed a drift in excess of 2% when responding to earthquake ground motions.

C2.7.4.2 Shared Element Condition

In many older urban areas, two buildings under different ownership often share in common the wall separating the two structures. These "party" walls often form part of the lateral and gravity load systems for both structures. If the buildings attempt to move independently during response to earthquakes, the shared wall can be pulled away from one or the other of the structures, resulting in partial collapse. Similar conditions often occur in buildings constructed with expansion joints. In such buildings, a single line of columns may provide gravity support for portions of both structures. Again, differential lateral movement of the two structures can result in collapse.

C2.7.4.3 Hazards from Adjacent Structures

There are a number of instances on record in which buildings have experienced life-threatening damage, and in some cases collapse, not as a result of their own inadequacies, but because debris or other hazards from an adjacent structure affected them. In many cases, there may be little that can be done to mitigate this problem. However, it is important to recognize the problem's existence and the consequences with regard to probable building earthquake performance. It makes little sense to rehabilitate a building to Enhanced Rehabilitation Objectives if it is likely to have an adjacent structure collapse on it. In such cases, the best seismic risk mitigation measure may be to relocate critical functions to another building.

C2.8 Rehabilitation Methods

Two basic methods for developing a rehabilitation design are defined in the *Guidelines*. These are Simplified Rehabilitation—a method available for some structures in which deficiencies common to certain model building types, and known to have caused poor earthquake performance in the past, are directly mitigated—and Systematic Rehabilitation, a method available for any building, in which a complete analysis of the structure is performed, and all elements and components critical to obtaining the desired Rehabilitation Objective are checked for adequacy to resist strength and deformation demands against specific acceptance criteria.

C2.8.1 Simplified Method

The Simplified Rehabilitation Method uses direct guidelines for mitigating specific types of deficiencies common to certain model buildings. They are based on the fact that for certain relatively simple types of structures, poor performance in earthquakes has repeatedly been observed to be the result of several critical failure modes, uniquely tied to the common construction detailing inherent in these model building types. Examples include light wood frame structures, which commonly experience partial collapse due to the presence of unbraced cripple walls; and reinforced and unreinforced masonry buildings and concrete tiltup buildings, which commonly experience partial collapse due to a lack of adequate out-of-plane attachment between the heavy walls and flexible diaphragms. The Simplified Rehabilitation Method provides specifications for direct remediation of these characteristic deficiencies, without necessarily requiring a complete numerical analysis of the building's lateral-force-resisting system. However, as a minimum, a complete evaluation in accordance with FEMA 178 (BSSC, 1992a) is recommended prior to specifying the Simplified Rehabilitation Method.

Most building structures, regardless of whether or not they have explicitly been designed for lateral-force resistance, do have both formal and informal lateral-force-resisting systems and, therefore, significant capability to resist limited levels of ground shaking without experiencing severe damage or instability. As an example, the architectural partitions in light wood frame construction together with the ceilings, floors, and roofs will typically form a complete lateral-force-resisting system with capacity to resist a significant portion of the building's weight, applied as a lateral force, even though few such structures have been designed for this behavior. Therefore, if the Simplified Rehabilitation guidelines for such structures are implemented, a structure with significant but unquantified seismic resistance will be obtained. If a FEMA 178 (BSSC, 1992a) evaluation is performed and all deficiencies identified in the evaluation are mitigated using the Simplified Rehabilitation Method, then the building is judged capable of achieving the Life Safety Performance Level for 10%/50 year ground shaking demands. However, because these procedures do not include a complete check of the adequacy of all important elements in the structures, and because the stability of the structure under larger levels of ground motion—or when subject to other hazards such as liquefaction or differential settlement—is not certain, Simplified Rehabilitation is not considered to achieve the BSO.

C2.8.2 Systematic Method

In Systematic Rehabilitation, a complete analysis of the adequacy of all important elements of the building to resist forces and deformations induced in the structure by its response to the ground motion and other earthquake hazards is conducted. Compared with procedures used in the design of new structures, greater attention is given to the effects of earthquake response on elements of the structure not specifically intended to be part of the lateral-force-resisting system. Any element that is critical to attainment of the desired performance level must be analyzed in Systematic Rehabilitation. This includes elements required to resist gravity loads, as well as nonstructural components that are important to the attainment of the performance.

C2.9 Analysis Procedures

Two basic analysis approaches for confirming the adequacy of a rehabilitation strategy are defined in the *Guidelines*. These are linear (elastic) analysis and nonlinear (inelastic) analysis. Both approaches may be performed using either static or dynamic procedures. The applicability of each of these procedures to a given structure is based on their ability to reasonably predict the likely distribution of seismic demands on the various structural elements and components that the building comprises. These issues are discussed below.

C2.9.1 Linear Procedures

In Linear Dynamic Procedures (LDP) and Linear Static Procedures (LSP), lateral forces are distributed to the various elements and components of the structure in accordance with their relative elastic stiffness characteristics. As in the *NEHRP Recommended Provisions,* FEMA 222A (BSSC, 1995), the lateral forces applied to the structure may be determined based upon a dynamic Time-History Analysis, a response spectrum method analysis, or a simplified equivalent static procedure based on the typical dynamic response of well-behaved, regular structures. While the linear procedures contained in the *Guidelines* are parallel to those contained in BSSC (1995) for new building design, the manner in which the forces and deformations predicted by these procedures are evaluated is significantly different.

The *NEHRP Recommended Provisions* for design of new structures attempt to control earthquake performance by requiring that buildings possess a minimum lateral-force-resisting strength and sufficient elastic stiffness to resist lateral forces within defined drift limits. The lateral forces used for design are based on an elastic analysis of the response of the structure to the design ground motion, but are scaled down substantially—by a response modification factor R—from the level that would be experienced by a structure with adequate strength to resist earthquake-induced forces within the elastic range. These response modification factors have been set based on the judgment and experience of those who wrote the building codes, and are based, to some extent, on the observed performance of buildings in past earthquakes. Use of these scaled-down forces in designing structures implies that when subjected to a design event, the structures will experience significant inelastic demands, and displacements will be substantially larger (by a factor C_d) than calculated under the specified design forces. Limitations on structural configuration, and special requirements for structural detailing and quality of materials, are included in the provisions in parallel with the strength requirements, so that the building may behave acceptably under these conditions.

The approach taken for new construction is not always directly applicable to existing buildings, which often have an unfavorable structural configuration, nonconforming detailing, and materials of substandard quality. Such a structure, even though provided with the minimum strength specified by the building codes for new construction, may not have adequate inelastic deformation capacity to resist the design earthquake within the desired performance limits. Therefore, the linear methods contained in the *Guidelines* have been specifically formulated to allow evaluation of the adequacy of the various building components to resist the inelastic deformation and strength demands which will be imposed on them by a design earthquake.

As with the *NEHRP Recommended Provisions*, an analysis is performed to determine the response (strength and deformation demands) that would be imposed on the structure by the design earthquake, if the building remained completely elastic. However, instead of reducing the earthquake forces by *R* and then combining them with other loads, the earthquake forces are directly combined with those imposed by dead and live loads and compared against the yield capacity of the components. If all critical actions of the components are found to have acceptable levels of capacity for the implied demands, as judged by the permissible values of a component ductility measure, *m*, specified in the materials chapters for the various Performance Levels, and the inter-story drifts predicted by the analyses are also within acceptable levels, then the rehabilitation design is deemed adequate. However, if some critical component actions are determined to have ductility demands that exceed acceptable levels, or if inter-story drifts are found to be excessive for the desired Performance Level, then the design is deemed inadequate.

When a linear procedure indicates that a rehabilitation design is inadequate for the desired performance levels, a number of alternatives are available. These include the following:

- If the inadequacy of the design is limited to a few primary elements (or components), it is possible to designate these deficient elements (or components) as secondary. The structure can then be reanalyzed and evaluated to determine if acceptable performance is predicted.
- If the analysis indicates only limited inadequacy, the use of a nonlinear procedure may demonstrate acceptable performance. This is because the nonlinear procedures provide more accurate estimates of demands than do linear procedures. This permits the use of somewhat more liberal acceptance criteria, resulting in some structures indicated as being marginal under linear procedures to be found to be acceptable by nonlinear procedures.
- The design can be revised to include additional rehabilitation measures that provide increased stiffening, strengthening, energy dissipation capacity, or response modification, or an alternative rehabilitation strategy can be selected.

Some structural components do not have significant inelastic deformation capacity. These brittle elements will fail if the load on them exceeds their capacity. An example is a column, which will buckle if loaded with excessive axial force. Such components could conservatively be evaluated in the linear procedures using a maximum permissible *m* value of 1.0. However, such an approach would often be too conservative. Because most elements in a structure have some ductility, and will respond in an inelastic manner in an earthquake, the unreduced force demands predicted on brittle components by a linear procedure may be substantially larger than those that the structure is actually capable of imposing on the component. To predict accurately the demands on such an element, a nonlinear procedure should be performed. In lieu of such a procedure, the linear procedures permit maximum strength demands on brittle elements to be estimated using an approximate force-delivery-reduction factor, designated *J*.

Linear procedures, while easy to apply to most structures, are most applicable to buildings that actually have sufficient strength to remain nearly elastic when subjected to the design earthquake demands, and buildings with regular geometries and distributions of stiffness and mass. To the extent that buildings analyzed by this method do not have such strength or regularity, the indications of inelastic ductility demands predicted by the elastic methods may be very inaccurate. In recognition of the relative inaccuracy of the linear techniques, the acceptance criteria contained in the materials chapters have intentionally been set with some level of conservatism, in order to provide a reasonable level of confidence that overall structural performance to the desired level can be attained.

Buildings that have relatively limited inelastic demands under a design earthquake may be evaluated with sufficient accuracy by linear procedures, regardless of their configuration. If the largest component DCR calculated for a structure does not exceed 2.0, the structure may be deemed to fall into this category, for the particular earthquake demand level being evaluated.

For buildings that have irregular distributions of mass or stiffness, irregular geometries, or nonorthogonal lateral-force-resisting systems, the distribution of demands predicted by an LDP analysis will be more accurate than those predicted by the LSP. Either the response spectrum method or Time-History Method may be used for evaluation of such structures.

Section 2.9.1 provides guidance as to when a dynamic procedure should be used.

A linear procedure is deemed applicable unless the results derived from the analysis indicate large ductility demands and the presence of certain irregularities, which would invalidate the predicted distribution of demands. The user must first determine whether an LSP or LDP should be used. An LDP may always be used, in those cases where linear procedures are applicable. The LSP may be used unless either vertical or torsional stiffness or mass irregularities exist. Stiffness or mass irregularities in a structure produce mode shapes that can be significantly different from those typical for a regular structure. Consequently, structures with these irregularities present may have substantially different responses to earthquake ground motion than regular structures. Since the lateral forcing function used in the LSP is derived from the response of regular structures, it should not be used for structures with these irregularities.

The presence of mass or stiffness irregularities, or both, can often be determined only after some analysis. The *Guidelines* suggest that if a user is in doubt with regard to the presence of such irregularities, the LSP may be employed to determine if such irregularities exist. The pattern of displacements in the structure predicted by such an analysis will typically indicate the presence of these irregularities. If a vertical stiffness or mass irregularity is present, this will typically show up as a concentration of drift demand in the structure. In vertically regular structures, inter-story drifts will be distributed in a uniform manner up the structure. In vertically irregular buildings, some stories will exhibit significantly greater drift than others. Similarly, if torsional stiffness or mass irregularities are present, the displacement pattern predicted by the LSP will indicate significant twisting of the structure, in plan.

In addition to being recommended for irregular structures, the LDP is also recommended for structures with heights that exceed 100 feet and buildings with nonorthogonal lateral-force-resisting systems. LDPs are recommended for tall structures because their response is often dominated by higher modes, which are more accurately tracked by the dynamic procedure. Also, tall buildings are generally important structures and warrant the extra care in modeling required to perform a dynamic procedure. Similarly, buildings with nonorthogonal lateral-force-resisting systems typically experience complex patterns of lateral movement (i.e., twisting and translation in directions that are skewed relative to the principal axes), resulting in element stresses and deformations that are more difficult to predict. For such buildings, the more careful development of an analytical model typically required for a dynamic procedure is deemed appropriate.

Once a linear procedure, either static or dynamic, has been performed for a structure, it is possible to determine if the predicted response is sufficiently elastic or uniform to justify the procedure's use. This is done by examining the distribution of calculated DCR values for the critical actions of the controlling components of the primary elements. The critical actions for a component are the independent "weak link" actions that can limit the participation of the component in the structural system.

Table C2-1 lists the typical actions for common structural components. The concept of "critical actions" will be demonstrated by example, in this case the components of a single bay reinforced concrete portal frame. The components are the columns, the beam, and the joint between each column and the beam. As indicated in Table C2-1, the various actions that can limit the beam's capacity to participate in the lateral-force-resisting system include its shear capacity and the flexural capacity of the section at either end for positive and negative bending moments. For each of these actions, a DCR value is calculated, based on the results of the linear procedure. First, the DCR values for the beam flexural capacity are calculated. Next, the beam is evaluated to determine whether it is shear critical or flexurally critical. The flexurally limited shear is calculated using Equation C2-2.

$$V_f = \frac{(M_L + M_R)}{L} + V_D + V_L \qquad \text{(C2-2)}$$

Table C2-1 Typical Actions for Structural Components

Structural Component	Action
Brace	Member axial force Connection axial force
Steel or Timber Beam or Column	Member axial force Member end shear force Member end moment Connection axial force Connection shear force Connection moment
Reinforced Concrete or Masonry Beam, Column, or Pier	Axial force End shear force End positive moment End negative moment Joint shear capacity
Unreinforced Masonry Pier or Spandrel	Axial force End shear force End moment

where:

L = Length of the beam span between points of plastic hinging

M_L = Plastic capacity of the beam at the left end

M_R = Plastic capacity of the beam at the right end

V_D = Beam shear due to dead loads

V_f = Shear resulting from development of the beam's plastic flexural capacity, at each end

V_L = Beam shear due to live loads

If the value of V_f is less than the nominal shear capacity of the beam, then the beam is flexurally critical and the controlling DCR values for bending at either end of the beam are the critical values. If V_f is greater than the beam's shear capacity, then the beam is shear critical and the DCR value computed for beam shear is the critical value for the component. Next, critical DCRs are determined for the other frame components, including the columns and the beam-column joints.

Determination of the controlling components for an element can be done by simple comparison of the calculated DCR values for the critical actions of each of the various components. The controlling component is the one that will reach its capacity at the lowest level of lateral loading to the element. The component with the highest calculated DCR value for its critical action will be the controlling component. If this frame were proportioned such that under increasing lateral loads the columns reached their capacity in flexure (or shear, or axial load) prior to the beam reaching its critical capacity, then the columns would be the controlling components. In this case, the calculated DCR values for the critical column components would exceed those for the beam.

C2.9.2 Nonlinear Procedures

Nonlinear procedures generally provide a more realistic indication of the demands on individual components of structures that are loaded significantly beyond their elastic range of behavior, than do linear procedures. They are particularly useful in that they provide for:

- More realistic estimates of force demands on potentially brittle components (force-controlled actions), such as axial loads on columns and braces
- More realistic estimates of deformation demands for elements that must deform inelastically in order to dissipate energy imparted to the structure by ground motions
- More realistic estimates of the effects of individual component strength and stiffness degradation under large inelastic demands
- More realistic estimates of inter-story drifts that account for strength and stiffness discontinuities that may develop during inelastic response
- Identification of critical regions in which large deformation demands may occur and in which particular care should be taken in detailing for ductile behavior
- Identification of strength discontinuities in plan or elevation that can lead to changes in dynamic characteristics in the inelastic range

Two nonlinear procedures are contained in the *Guidelines*. These are a simplified Nonlinear Static Procedure (NSP) and a more detailed Nonlinear Dynamic Procedure (NDP). Nonlinear procedures may be used in the rehabilitation analysis of any structure. They should be used whenever the results of a linear procedure indicate that DCRs for critical actions of primary components are substantially in excess of 2.0, and in particular, when the distribution of these inelastic demands throughout the structure is nonuniform. An irregular distribution of DCRs based on a linear procedure indicates that the structure has the potential to form inelastic soft stories, or inelastic torsional instabilities. When such conditions exist, elastic analyses cannot predict the distribution of earthquake demands with any accuracy. A nonlinear procedure should be used in these cases.

C2.9.2.1 Nonlinear Static Procedure (NSP)

This static, sequential nonlinear procedure approach avoids many of the inaccuracies inherent in the linear methods by permitting direct, although approximate, evaluation of the inelastic demands produced in the building by the design earthquake. As with the linear procedures, a mathematical model of the building, representing both the existing and new elements, is constructed. However, instead of performing an elastic analysis of the response of the structural model to specified ground motion, an incremental nonlinear analysis is conducted of the distribution of deformations and stresses throughout the structure as it is subjected to progressively increased lateral displacements. Acceptance criteria include permissible deformation (for example, elongations, drifts, and rotations) and strength demands on common elements and components for different Performance Levels. By comparing the results of the incremental force-displacement analysis (“pushover”) with these acceptance criteria, it is possible to estimate limiting overall structural displacements at which each desired Structural Performance Level can be achieved. Overall displacement demands likely to be produced on the structure by the design earthquake(s) are then approximated using simplified general relationships between elastic spectral response and inelastic response. These relationships take into account, in an approximate manner, the effects of period lengthening, hysteretic damping, and soil structure interaction.

The NSP is generally a more reliable approach to characterizing the performance of a structure, at a given level of excitation, than are the linear procedures. However, it is not an exact approach. It cannot accurately account for the changes in dynamic response and in inertial load patterns that develop in a structure as it degrades in stiffness. Further, it cannot account for the effects of higher mode response in an accurate manner. For this reason, the *Guidelines* recommend that when the NSP is utilized on a structure that has significant higher mode participation in its response, the LDP should also be employed to verify the adequacy of the design. When this approach is taken, somewhat less restrictive criteria are permitted for the LDP than are normally associated with its use, recognizing the significantly improved knowledge of the building’s probable seismic response that is obtained by performing both analysis procedures.

Despite the above-noted limitations on the accuracy of the NSP, it is still generally considered to provide a better estimate of the probable performance of structures than the linear procedures alone. The inelastic force and displacement demands on structural components are directly—albeit approximately—calculated. Therefore, when using this approach it is possible to directly use test data contained in the literature or performed on a project-specific basis to set permissible levels of demand, rather than relying on the less accurately developed *m* values used as acceptance criteria in the linear procedures.

Since the nonlinear procedures more accurately predict demands on individual components than do the linear procedures, acceptance criteria have been developed with less inherent margin. Accordingly, it is expected that the application of this technique will often result in rehabilitation designs that require less remedial work to the building than do the linear procedures. Consequently, the nonlinear procedures are an excellent way to conduct the more detailed evaluations of a building suggested in FEMA 178 (BSSC, 1992a).

Although only a single Nonlinear Static Procedure (NSP) is presented in the *Guidelines*, a number of related approaches are currently in use. These include the Capacity Spectrum Method (Department of the Army, Navy, and Air Force, 1986) and the Secant Modulus Method (Kariotis et al., 1994). Several of these approaches can estimate the effects of higher modes and changing patterns of inertial forces at increasing response more easily than does the NSP. Such methods may provide more accurate evaluations of probable building response for some structures.

C2.9.2.2 Nonlinear Dynamic Procedure (NDP)

The NDP consists of nonlinear Time-History Analysis, a sophisticated approach to examining the inelastic demands produced on a structure by a specific suite of ground motion time histories. As with the NSP, the results of the NDP can be directly compared against test data on the behavior of representative structural components in order to identify the structure’s probable performance when subjected to a specific ground motion. Potentially, the NDP can be more accurate than the NSP in that it avoids some of the approximations made in the more simplified analysis. Time-History Analysis automatically accounts for higher mode effects and shifts in inertial load patterns as structural softening occurs. In addition, for a given earthquake record, this approach directly solves for the maximum global displacement demand produced by the earthquake on the structure, eliminating the need to estimate this demand based on general relationships.

Despite these advantages, it is believed that the NDP is currently limited in application for a number of reasons. First, currently available computer hardware and software effectively limit the size and complexity of structures that may be analyzed by this technique. At present, there is no general-purpose nonlinear analysis software that will permit practical evaluation of large structures that include elements with the wide range of inelastic constitutive relations actually present in the building inventory. Further, these analyses tend to be highly sensitive to small changes in assumptions with regard to either the character of the ground motion record used in the analysis, or the nonlinear stiffness behavior of the elements. As an example, two ground motion records enveloped by the same response spectrum can produce radically different results with regard to the distribution and amount of inelasticity predicted in the structure.

It is expected that the limitations of software and hardware available to perform these analyses will eventually be resolved. However, sensitivity of the analyses to basic assumptions will remain a problem. In order to reliably apply this approach to rehabilitation design, it is necessary to perform a number of such analyses, using varied assumptions. The sensitivity of the analysis approach to the assumptions incorporated is the principal reason why this method should be used only for projects for which independent review is provided by qualified third-party experts.

The NSP is generally applicable to most building configurations and rehabilitation strategies. The NDP is also suitable for general application, although independent third-party review is recommended.

C2.9.3 Alternative Rational Analysis

During the development of the *Guidelines*, a number of existing analytical techniques for use in seismic rehabilitation design—as well as some that were under development—were evaluated for their applicability to the *Guidelines*. Many of these were found to be applicable to only specific Model Building Types and others to only one Rehabilitation Objective, often different from those contained in the *Guidelines*. Rather than adopting and modifying a number of these individual procedures, the *Guidelines* writers chose to develop the four general-purpose procedures (Linear Static, Linear Dynamic, Nonlinear Static, Nonlinear Dynamic) contained in the *Guidelines* and make them broadly applicable to all Model Building Types and Rehabilitation Objectives. These general-purpose procedures are based largely on many of these other preexisting approaches as well as some under parallel development. The fact that a specific rehabilitation procedure has not been adopted verbatim into the *Guidelines* should not be taken as an indication that the procedure is invalid or should not be used. Such procedures may continue to be used; however, it should not be assumed, without thorough review, that the specific Rehabilitation Objectives of the *Guidelines* may be attained through the use of these alternative procedures.

It is anticipated that as computing technology and the knowledge of structural behavior improve, additional procedures will become available that some engineers will desire to use in seismic rehabilitation. Such use is encouraged. However, independent expert review is recommended as a condition of such use because, like all developmental approaches, these procedures may be limited in applicability; may lead to inappropriate designs in some instances; and may not be developed to a sufficient level of detail for general application. When applying alternative analytical procedures, special caution is advised with regard to the adoption of the acceptance criteria contained in the *Guidelines*. The acceptance criteria contained in the *Guidelines* are specifically intended for use with the analytical procedures contained in the *Guidelines*, and may produce incorrect or meaningless results when applied to alternative analytical approaches.

C2.9.4 Acceptance Criteria

No commentary is provided for this section.

C2.10 Rehabilitation Strategies

The rehabilitation strategy is the basic approach used in mitigating the deficiencies previously identified in the structure. In Simplified Rehabilitation, the strategy is one of mitigating deficiencies relative to FEMA 178 (BSSC, 1992a), often by highly prescriptive techniques, as for example a requirement that sill plates be bolted to foundations. However, in Systematic Rehabilitation, a wide range of strategies may be available, depending on the nature of the specific deficiencies involved. For a given building and set of Rehabilitation Objectives, some strategies will be more or less effective than others, and can result in widely different rehabilitation costs. Complete discussion of the alternative strategies available is beyond the scope of this document; however, the publication *NEHRP Handbook of Techniques for the Seismic Rehabilitation of Buildings* (BSSC, 1992b), provides good background material.

The *Guidelines* allude to the importance of providing redundancy in a structure's lateral-force-resisting system but provide no direct method to evaluate whether sufficient redundancy is present in a structure. Recently adopted codes for new buildings, including the 1997 *Uniform Building Code* (ICBO, 1997) and the *NEHRP Recommended Provisions* (BSSC, 1997) have adopted a specific redundancy coefficient, ρ, that is used to adjust the design seismic forces based on the percentage of the total lateral force resisted by any single component in the structure. This coefficient varies from a value of 1.0, for highly redundant structures, to a value of 1.5 for structures with very limited redundancy. The effect of this coefficient is to provide greater margin against failure for structures that rely heavily on the resistance provided by only a few elements. This concept was not specifically adopted by the *Guidelines*. However, it may be worth considering, particularly when rehabilitating buildings with nonredundant systems. The ρ coefficients adopted by the 1997 *UBC* (ICBO, 1994) and *NEHRP Recommended Provisions* (BSSC, 1997) documents could be directly used with the *Guidelines* to account for redundancy effects in an explicit, if not rigorous manner. For the linear procedures, this could be done by directly multiplying the base shear forces by the ρ coefficient. For the NSP, this could be done by multiplying the target displacement by this coefficient. For the NDP, it would be necessary to multiply the ground motion records by the coefficient.

C2.11 General Analysis and Design Requirements

This section provides guidelines for controlling important seismic performance attributes, such as continuity and interconnection of elements, that are not directly evident as potential deficiencies from an analytical evaluation. The requirements are mostly based on parallel provisions contained in the *NEHRP Provisions*.

C2.11.1 Directional Effects

This section requires that a building be demonstrated to be capable of resisting ground motion incident from any direction. For structures that are rectangular or nearly rectangular in plan, analysis of building response about the two principal orthogonal building axes is sufficient. For buildings of unusual shape, analyses of building response to applied ground motion incident from other directions may be required.

C2.11.2 P-Δ Effects

Earthquake-induced collapse of buildings that experience excessive drift can occur as a result of secondary stresses attributable to the P-Δ effect. Equation 2-14 in the *Guidelines* uses a first-order linear approximation of P-Δ effects. More accurate approaches, directly incorporating elastic stability theory, could also be employed.

C2.11.3 Torsion

The effects of torsion are much more important to seismic performance than they are to wind resistance. Engineers familiar with wind design but not with seismic design may overlook torsional effects by utilizing two-dimensional analysis techniques. This section reminds the engineer of the importance of capturing torsional behavior in the analysis.

C2.11.4 Overturning

In addition to creating lateral shear forces in structures, earthquake ground motion also results in a tendency for structures, and individual vertical elements of structures, to overturn about their bases. Although actual overturning of structures due to earthquake ground motion is very rare, overturning effects do have the potential to result in significant stresses in structures, which have caused local and even global failures. In the design of new buildings, earthquake effects, including overturning, are evaluated for lateral forces that are much lower (reduced by the factor *R*) than those the structure actually will experience. The designer typically evaluates the effects of overturning in one of two ways:

1. For elements that are provided with positive attachment between levels, such as reinforced concrete or masonry shear walls, or moment-resisting frames, the overturning effects are resolved into component forces, e.g., flexure at the base of a wall pier; and the component is then proportioned with adequate strength to resist these overturning effects at the reduced force levels.

2. Some elements, such as wood shear walls and foundations, may not be provided with positive attachment to lower levels. For these elements, an overturning stability check is typically performed. If the element supports sufficient dead load to remain stable under the overturning effects of the design lateral forces, then the design is deemed adequate. However, if it is determined that the element has inadequate dead load to remain stable against overturning, then hold-downs, piles, or other types of uplift anchors are provided to resist overturning effects.

In the linear procedures contained in the *Guidelines*, the lateral forces used to evaluate the performance of a structure have not been reduced by the *R*-factor, as they typically are in the design of new buildings. As a result, the computed effects of overturning will be more severe, if calculated in the typical manner, than is the case during the design of new buildings. Though the procedure used to design new buildings for earthquake-induced overturning is not completely rational, it has resulted in successful performance. Therefore, it was felt that it would be inappropriate for the *Guidelines* to require that structures and elements of structures remain stable for the full lateral forces used in the linear procedures. Instead, just as with new buildings, the designer must determine if positive direct attachment will be needed to resist overturning effects, or alternatively, if sufficient dead load is present on the element to resist these effects. If dead loads are used to resist overturning without supplemental positive direct attachment, then overturning is treated as a force-controlled behavior and the overturning demands are reduced to an estimate of the real overturning demands that can be transmitted to the element, considering the overall limiting strength of the structure. As with the design of new buildings, a stability evaluation is performed, and in addition, the element is evaluated for adequacy to resist bearing stresses at the toe, about which it is being overturned.

If it is determined that there is inadequate dead load on an element to resist overturning effects, then positive structural attachment must be provided to resist overturning effects. Examples of such attachment included piles or caissons with uplift anchors at foundations; dowels or reinforcing that extends between the boundary elements of a shear wall at one level to that in the level below; and hold-down hardware attached to the end stud of a timber shear wall in one level and that in the level below. The individual materials chapters provide guidance as to whether each of these elements is to be treated as deformation-controlled or force-controlled for evaluation and design purposes.

When nonlinear procedures are performed, the effects of overturning can be directly investigated in the mathematical model. This is accomplished by releasing the rotational restraint on elements, once the demands on the elements exceed the stabilizing forces. One of the principal benefits of the nonlinear procedures is that they permit a more realistic evaluation of overturning effects than do the linear procedures.

C2.11.5 Continuity

A continuous structural system with adequately interconnected elements is one of the most important prerequisites for acceptable seismic performance. The requirements of this section are similar to parallel provisions contained in the BSSC (1995) provisions.

C2.11.6 Diaphragms

The concept of a diaphragm chord, consisting of an edge member provided to resist diaphragm flexural stresses through direct axial tension or compression, is not familiar to many engineers. Buildings with solid structural walls on all sides often do not require diaphragm chords. However, buildings with highly perforated perimeter walls do require these components for proper diaphragm behavior. This section of the *Guidelines* requires that these components be provided when appropriate.

A common problem in buildings that nominally have robust lateral-force-resisting systems is a lack of adequate attachment between the diaphragms and the vertical elements of the lateral-force-resisting system to affect shear transfer. This is particularly a problem in buildings that have discrete shear walls or frames as their vertical lateral-force-resisting elements. This section provides a reminder that it is necessary to detail a formal system of force delivery from the diaphragm to the walls and frames.

Diaphragms that support heavy perimeter walls have occasionally failed in tension induced by out-of-plane forces generated in the walls. This section is intended to ensure that sufficient tensile ties are provided across diaphragms to prevent such failures. The design force for these tensile ties, taken as $0.4S_{XS}$ times the weight, is an extension of provisions contained in the 1994 *Uniform Building Code* (ICBO, 1994). In that code, parts and portions of structures are designed for a force calculated as C_pIZ times the weight of the component with typical values of C_p being 0.75 and Z being the effective peak ground acceleration for which the building is designed. The 1994 UBC provisions use an allowable stress basis. The *Guidelines* use a strength basis. Therefore, a factor of 1.4 was applied to the C_p value, and a factor of 1/(2.5) was applied to adjust the Z value to an equivalent S_{XS} value, resulting in a coefficient of 0.4.

C2.11.7 Walls

Inadequate anchorage of heavy masonry and concrete walls to diaphragms for out-of-plane inertial loads has been a frequent cause of building collapse in past earthquakes. Following the 1971 San Fernando earthquake, the *Uniform Building Code* adopted requirements for positive direct connection of wall panels to diaphragms, with anchorage designed for a minimum force equal to ZIC_pW_p. In this equation, the quantity ZIC_p represents the equivalent out-of-plane inertial loading on the wall panel and typically had a value that was 75% of the effective peak ground acceleration for the site. This section of the *Guidelines* imposes design provisions based on observations made following the 1994 Northridge earthquake. Failures occurred in a number of buildings meeting the requirements of the building code in effect at that time. Actual strong motion recordings in buildings with flexible diaphragms indicates that these diaphragms amplify the effective peak ground accelerations by as much as three times. For a site with an effective peak horizontal ground acceleration of 0.4g (S_{XS} = 1.0g), this would correspond to an inertial acceleration of the wall panels of 1.2g. The χ coefficients contained in Table 2-18 were derived from this relationship, providing for somewhat greater factors of safety at the Immediate Occupancy Performance Level and reduced factors of safety at the Collapse Prevention Performance Level. More thorough treatment of this subject may be found in Hamburger and McCormick (1994).

These failures also extended to walls of construction other than concrete and masonry, even though earthquake-induced collapse of such walls is rare. This can be considered a matter of collateral rehabilitation for wind-load resistance. Lack of adequate out-of-plane anchorage for wood stud walls has occasionally resulted in failures in tornadoes and high wind storms. Use of the *Guidelines* will reduce the vulnerability of wood buildings to such failures.

C2.11.8 Nonstructural Components

There is a tendency for structural engineers to address structural deficiencies but neglect nonstructural problems, which can have life safety implications as well important economic implications. This section serves as a reminder of the importance of addressing these issues.

C2.11.9 Structures Sharing Common Elements

Structures that share elements in common are particularly problematic. Where practical, the best approach for such structures may be to tie the buildings together, such that they behave as one structure. Alternate approaches could include ensuring that differential displacements of the two structures cannot result in a collapse condition, or providing redundant structural elements such that if failure of the shared element occurs, stability is still maintained.

C2.11.10 Building Separation

Buildings that have inadequate separation can impact each other, or “pound” during response to ground motion. This can drastically alter the buildings’ performance and should be considered in rehabilitation design. The first step is to determine if pounding is likely to occur. One approach to determining the likelihood of pounding is to take the absolute sum of the expected lateral deflections of each building at the location of potential impacts, and if the available separation of the buildings is greater than this amount, assume that pounding does not occur. The implicit assumption in such an approach is that at some point during the buildings' response to the ground motions, the structures will become completely out of phase and require a separation of the calculated amount.

An alternative approach to evaluating the potential for pounding, termed the spectral difference approach (Jeng et al., 1992), directly accounts for the incoherence of multimode response, and the fact that both structures are unlikely to experience the maximum response of all modes at the same instant, completely out of phase. This approach requires knowledge of the natural modes of both structures. Since such information is often not available for one of the structures, the *Guidelines* adopt a somewhat simpler approach of using a square root of the sum of the squares (SRSS) combination of estimated structural lateral deflections to check the adequacy of building separation. This approach requires only an estimate of the lateral deflection of the adjacent structure (which can be based on general rules of thumb), rather than performance of a modal analysis on each structure. However, it accounts for the fact that some incoherence of response is likely to occur and permits less than the full separation required if both structures are assumed to behave completely out of phase.

When two adjacent structures pound, this can drastically alter the dynamic response of both structures, resulting in a change in the effective mode shapes and period of each, as well as the pattern and magnitude of inertial demands and deformations induced on both structures. The *Guidelines* permit buildings rehabilitated to the BSO to experience pounding as long as the effects of such pounding are adequately accounted for in the design.

Approximate methods of accounting for these effects can be obtained by performing nonlinear Time-History Analyses of both structures (Johnson et al., 1992). Approximate elastic methods for evaluating these effects have also been developed (Kasai et al., 1990) and are presented in the literature.

One of the most dangerous aspects of pounding is the potential for local destruction of critical structural components at the point of impact. As an example, the floor slabs of one structure can create a knife-edge effect against the columns of an adjacent structure, resulting in potential for partial or total collapse. Where such behavior is plausible, consideration should be given to altering the response of both structures such that impacts do not occur, or providing redundant elements at a location away from the zone of impact to replace components that may fail due to the impact effects.

Buildings that are likely to experience significant pounding should not be considered to be capable of meeting Enhanced Rehabilitation Objectives. This is because significant local crushing of building components is likely to occur at points of impact. Further, the very nature of the impact is such that high-frequency shocks can be transmitted through the structures and potentially be very damaging to architectural elements, and mechanical and electrical systems. Such damage is not consistent with the performance expected of buildings designed to Enhanced Rehabilitation Objectives.

C2.12 Quality Assurance

This section indicates the minimum construction quality assurance (QA) measures that should apply to any seismic rehabilitation project, regardless of the Rehabilitation Objectives, project complexity, or costs. The intent of these requirements is to assure that those resources invested in seismic rehabilitation result in the intended improvement in seismic reliability. Failure to properly implement rehabilitation measures can result in no improvement in the existing building's seismic resistance, or worse, a lessening of its resistance. For some projects that are highly complex, use unusual technologies, have exacting construction tolerance requirements, or are intended to achieve Enhanced Rehabilitation Objectives, it may be appropriate to implement measures beyond those contained in the *Guidelines*. The structural design professional of record should establish these on a project-specific basis.

C2.12.1 Construction Quality Assurance Plan

The development of a Quality Assurance Plan (QAP) is the only design period quality assurance measure specifically prescribed by the *Guidelines*; however, it is not the only design period quality assurance measure that should be taken. In addition to development of a QAP, the design professional should also take a number of other precautions to maintain the quality of the project. These include ensuring that:

- An adequate understanding of the existing construction characteristics of the structure has been developed, prior to embarking on a rehabilitation design.

- The construction documents adequately represent the intent of the design calculations and analyses, and these analyses and calculations are accurate.
- The construction documents are clear with regard to the existing conditions of the structure and the modifications that are to be made to it as part of the rehabilitation work.
- The construction documents specify the construction of details that are constructible, and specify the use of materials and methods that can be readily performed to attain the desired results.

These measures are not specified in the *Guidelines*, as they are a function of individual design office practice. However, they are an important part of any project.

C2.12.2 Construction Quality Assurance Requirements

C2.12.2.1 Requirements for the Structural Design Professional

In addition to other inspections and observations that may be made during the construction period, the design professional in responsible charge of development of the seismic evaluation, analyses, and rehabilitation design for the building should make site observations during the construction process. This is even more important in rehabilitation construction than it is in new construction. Often it is not practical to fully investigate the existing structural conditions of a building during the rehabilitation design. Consequently, when selective demolition of finishes occurs during the construction period, it is commonly found that the configuration, condition, and strength of some components of the existing building are significantly different than assumed in the rehabilitation design. It is imperative that the design professional become aware of any such deviations from the design assumptions so that the validity of detailing contained on the construction drawings, and perhaps the overall design, can be confirmed or adjusted as appropriate. Adjustments that may be necessary can range from minor revisions of individual details to complete alteration of the design concept.

Structural observation by the design professional is also extremely important in rehabilitation projects because many of the details used for rehabilitation construction can be significantly different from those commonly used in the construction of new buildings. Therefore, there is somewhat greater potential for construction error in the implementation of the details. Structural observation is an important tool for assuring that construction work is performed in accordance with the design intent.

C2.12.3 Regulatory Agency Responsibilities

No commentary is provided for this section.

C2.13 Alternative Materials and Methods of Construction

This section provides guidance for developing appropriate data to evaluate construction materials and detailing systems not specifically covered by the *Guidelines*. The *Guidelines* specify stiffnesses, *m* coefficients, strength capacities, and deformation capacities for a wide range of element and component types. To the extent practical, the *Guidelines* have been formatted to provide broad coverage of the various common construction types present in the national inventory of buildings. However, it is fully anticipated that in the course of evaluating and rehabilitating existing buildings, construction systems and component detailing practices that are not specifically covered by the *Guidelines* will be encountered. Further, it is anticipated that new methods and materials, not currently in use, will be developed that may have direct application to building rehabilitation. This section provides a method for obtaining the needed design parameters and acceptance criteria for elements, components, and construction details not specifically included in the *Guidelines*.

The approach taken in this section is similar to that used to derive the basic design parameters and acceptance criteria contained in the *Guidelines* for various elements and components, except that no original experimentation was performed. The required story-force deformation curves were derived by the *Guidelines* developers, either directly from research testing available in the literature, or based on the judgment of engineers knowledgeable in the behavior of the particular materials and systems.

C2.13.1 Experimental Setup

The *Guidelines* suggest performing a minimum of three separate tests of each unique component or element. This is because there can be considerable variation in the results of testing performed on "identical" specimens, just as there is inherent variability in the behavior of actual components and structural elements in buildings. The use of multiple test data allows some of the uncertainty with regard to actual behavior to be defined.

A specific testing protocol has not been recommended by the *Guidelines*, as selection of a suitable protocol is dependent on the anticipated failure mode of the assembly as well as the character of excitation it is expected to experience in the real structure. In one widely used protocol (ATC, 1992), the specimen is subjected to a series of quasi-static, fully reversed cyclic displacements that are incremented from displacement levels corresponding to elastic behavior, to those at which failure of the specimen occurs. Other protocols that entail fewer or greater cycles of displacement, and more rapid loading rates, have also been employed. In selecting an appropriate test protocol, it is important that sufficient increments of loading be selected to characterize adequately the force-deformation behavior of the assembly throughout its expected range of performance. In addition, the total energy dissipated by the test specimen should be similar to that which the assembly is anticipated to experience in the real structure. Tests should always proceed to a failure state, so that the margin against failure of the assembly in service can be judged.

If the structure is likely to be subjected to strong impulsive ground motions, such as those that are commonly experienced within a few kilometers of the fault rupture, consideration should be given to using a protocol that includes one or more very large displacements at the initiation of the loading, to simulate the large initial response induced by impulsive motion. Alternatively, a single monotonic loading to failure may be useful as a performance measure for assemblies representing components in structures subject to impulsive motion.

C2.13.2 Data Reduction and Reporting

It is important that data from experimental programs be reported in a uniform manner so that the performance of different subassemblies may be compared. The data reporting requirements specified in the *Guidelines* are the minimum thought to be adequate to allow development of the required design parameters and acceptance criteria for the various Systematic Rehabilitation Procedures. Some engineers and researchers may desire additional data from the experimentation program to allow calibration of their analytical models and to permit improved understanding of the probable behavior of the subassemblies in the real structure.

C2.13.3 Design Parameters and Acceptance Criteria

The *Guidelines* provide a multistep procedure for developing design parameters and acceptance criteria for use with both the linear and nonlinear procedures. The basic approach consists of the development of an approximate story lateral-force-deformation curve for the subassembly, based on the experimental data.

In developing the representative story lateral-force-deformation curve from the experimentation, use of the "backbone" curve is recommended. This takes into account, in an approximate manner, the strength and stiffness deterioration commonly experienced by structural components. The backbone curve is defined by points given by the intersection of an unloading branch and the loading curve of the next load cycle that goes to a higher level of displacement, as illustrated in Figure C2-5.

C2.14 Definitions

No commentary is provided for this section.

C2.15 Symbols

No commentary is provided for this section.

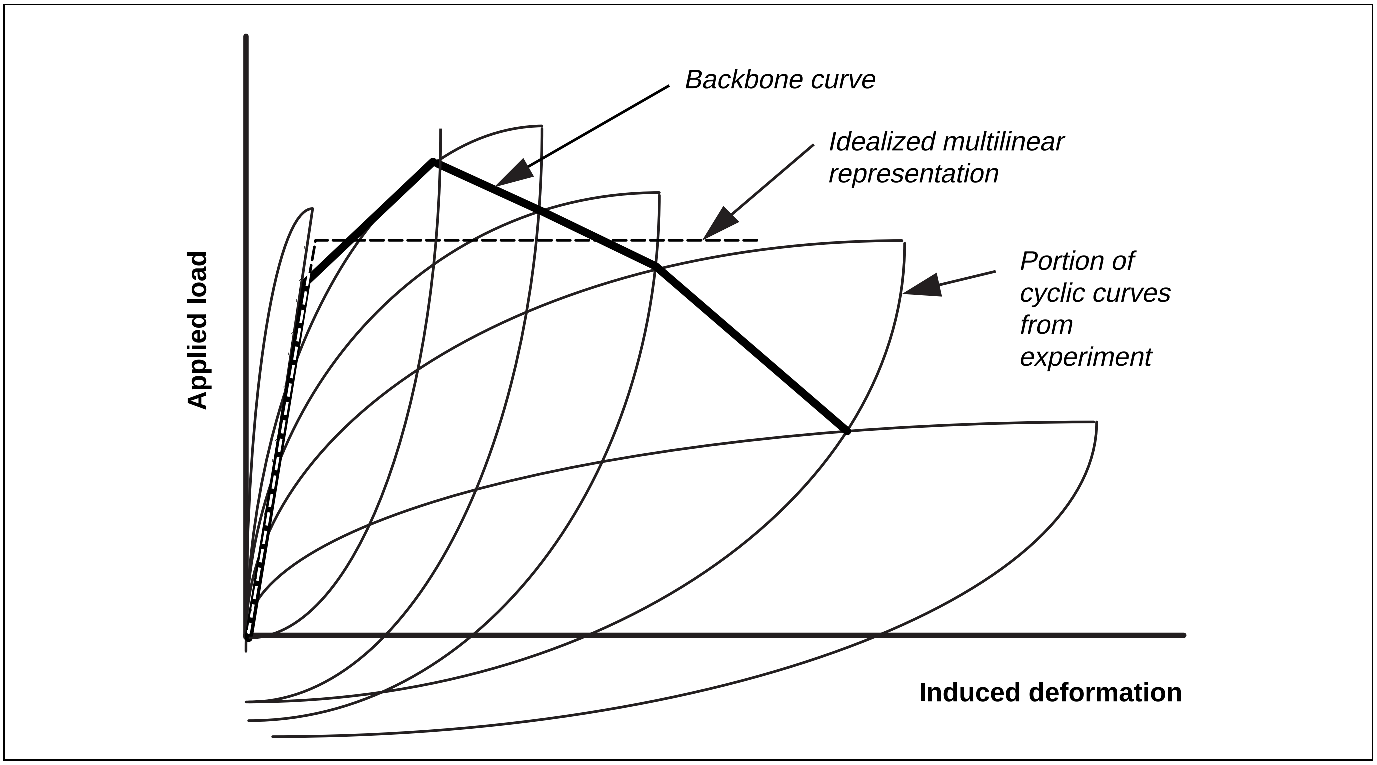


Figure C2-5 Idealized Force versus Displacement Backbone Curve

C2.16 References

ATC, 1978; second printing, 1984; *Tentative Provisions for the Development of Seismic Regulations for Buildings*, Report No. ATC-3-06, Applied Technology Council, Redwood City, California.

ATC, 1992, *Guidelines for Seismic Testing of Components of Steel Structures*, Report No. ATC-24, Applied Technology Council, Redwood City, California.

Boatwright, J., 1994, "Modeling Ground Motions in the Near Field of Rupturing Faults," *Proceedings: Seminar on New Developments in Earthquake Ground Motion Estimation and Implications for Engineering Design Practice*, Report No. ATC-35-1, Applied Technology Council, Redwood City, California, pp. 9-1–9-17.

BOCA, 1993, *National Building Code*, Building Officials and Code Administrators International, Country Club Hill, Illinois.

Boore, D. M., and Joyner, W. B., 1994, "Prediction of Ground Motion in North America," *Proceedings: Seminar on New Developments in Earthquake Ground Motion Estimation and Implications for Engineering Design Practice,* Report No. ATC-35-1, Applied Technology Council, Redwood City, California, pp. 6-1–6-41.

BSSC, 1992a, *NEHRP Handbook for the Seismic Evaluation of Existing Buildings*, 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.

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.

Department of Army, Navy, and Air Force, 1986, *Seismic Design of Essential Facilities*, TM 5-810-10-1, Washington, D.C.

Dobry, R., Idriss, I. M., and Ng, E., 1978, "Duration Characteristics of Horizontal Components of Strong Ground Motion Earthquake Records," *Bulletin of the Seismological Society of America*, Vol. 68, No. 5, pp. 1487–1520.

Hamburger, R.O., and McCormick, D.L., 1994, "Implications of the January 17, 1994 Northridge Earthquake on Tiltup and Masonry Buildings with Wood Roofs," *Proceedings of the Structural Engineers Association of California 63rd Annual Convention,* SEAOC, Sacramento, California.

Heaton, T. H., and Hartzell, S. H., 1994, "Earthquake Ground Motions in the Near-source Region," *Proceedings: Seminar on New Developments in Earthquake Ground Motion Estimation and Implications for Engineering Design Practice*, Report No. ATC-35-1, Applied Technology Council, Redwood City, California, pp. 8-1–8-28.

Heaton, T. H., Hall, J. F., Wald, D. J., and Halling, M. W., 1995, "Response of High Rise and Base Isolated Buildings to a Hypothetical M_w 7.0 Blind Thrust Earthquake," *Science*, Vol. 267, pp. 206–211.

ICBO, 1994, *Uniform Building Code*, International Conference of Building Officials, Whittier, California.

ICBO, in preparation, *Uniform Building Code*, International Conference of Building Officials, Whittier, California.

Jeng, V., Kasai, K., and Maison, B. F., 1992, "A Spectral Difference Method to Estimate Building Separations to Avoid Pounding," *Earthquake Spectra*, Earthquake Engineering Research Institute, Oakland, California, Vol. 8, No. 2.

Johnson, J. J., Conoscente, J. P., and Hamburger, R. O., 1992, "Dynamic Analysis of Impacting Structural

Systems," *Proceedings of the Tenth World Conference on Earthquake Engineering*, Madrid, Spain.

Kariotis, J. C., Hart, G., Youssef, N., Guh, J., Hill, J., and Nglem, D., 1994, *Simulation of Recorded Response of Unreinforced Masonry (URM) Infill Frame Building*, SMIP 94-05, California Division of Mines and Geology, Sacramento, California.

Kasai, K., Maison, B. F., and Patel, D. J., 1990, "An Earthquake Analysis for Buildings Subjected to a Type of Pounding," *Proceedings of the Fourth U.S. National Conference on Earthquake Engineering*, Earthquake Engineering Research Institute, Oakland, California.

Martin, G., and Dobry, R., 1994, "Earthquake Site Response and Seismic Code Provisions," *Research Accomplishments, 1986–1994*, National Center for Earthquake Engineering Research, State University of New York at Buffalo, New York, pp. 121–129.

Newmark, N. M., and Hall, W. J., 1982, *Earthquake Spectra and Design*, Earthquake Engineering Research Institute, Oakland, California.

Rinne, E. E., 1994, "Development of New Site Coefficients for Building Codes," *Proceedings: Fifth U.S. National Conference on Earthquake Engineering*, Earthquake Engineering Research Institute, Oakland, California, Vol. III, pp. 69–78.

Sadigh, D., Chang, C. Y., Abrahamson, N. A., Chiou, S. J., and Power, M. S., 1993, "Specification of Long-period Ground Motions; Updated Attenuation Relationships for Rock Site Conditions and Adjustment Factors for Near-Fault Effects," *Proceedings, Seminar on Seismic Isolation, Passive Energy Dissipation and Active Control*, Report No. ATC-17-1, Applied Technology Council, Redwood City, California.

SBCCI, 1994, *Standard Building Code*, Southern Building Code Congress International, Birmingham, Alabama.

Sommerville, P., and Graves, R., 1993, "Conditions That Give Rise to Unusually Large Long-Period Ground Motions," *Proceedings, Seminar on Seismic Isolation, Passive Energy Dissipation and Active Control*, Report No. ATC-17-1, Applied Technology Council, Redwood City, California.

VSP, 1992, *A Benefit-Cost Model for the Seismic Rehabilitation of Buildings*, prepared by VSP Associates for the Building Seismic Safety Council and the Federal Emergency Management Agency (Report No. FEMA 227), Washington, D.C.

C3. Modeling and Analysis (Systematic Rehabilitation)

C3.1 Scope

Section 3.1 provides a road map for the user of Chapter 3. Much information relevant to the provisions of Chapter 3 can be found in Chapters 2, 5, 6, 7, and 8; the relationship of the provisions in Chapter 3 to those in other chapters is summarized in Section 3.1. The reader should be familiar with the relevant information presented in these chapters before implementing the analysis methods presented in Chapter 3.

The *Guidelines* present strategies for both Systematic Rehabilitation and Simplified Rehabilitation. The procedures in Chapter 3 are applicable only to the Systematic Rehabilitation Method.

C3.2 General Requirements

C3.2.1 Analysis Procedure Selection

Chapter 3 provides guidance for implementation of the *Guidelines*' four Analysis Procedures for systematic rehabilitation of buildings. Guidance on selection of the appropriate Analysis Procedure is presented in Chapter 2.

In the Linear Static Procedure (LSP) and the Linear Dynamic Procedure (LDP), the term "linear" implies "linearly elastic." However, geometric nonlinearities associated with gravity loads acting through lateral displacements may be included in the analysis model. Furthermore, components of concrete and masonry may be modeled using cracked-section properties, so that some material nonlinearity is modeled, even though the numerical analysis assumes perfectly linear behavior. In the Nonlinear Static Procedure (NSP) and the Nonlinear Dynamic Procedure (NDP), the term "nonlinear" refers to material nonlinearities (inelastic material response); geometric nonlinearities may also be considered.

C3.2.2 Mathematical Modeling

C3.2.2.1 Basic Assumptions

The *Guidelines* promote the use of three-dimensional mathematical models for the systematic rehabilitation analysis of buildings, but were written recognizing that fully three-dimensional modeling is not always feasible given available analysis tools, especially those for nonlinear analysis. Therefore, three-dimensional models are required only in certain cases known to require such models.

Where two-dimensional models are used, the model should be developed recognizing the three-dimensional nature of the building structure. For example, shear walls and other bracing systems commonly have cross sections that form "L," "T," and other three-dimensional shapes. Strength and stiffness of a "T"-shaped wall should be developed including the effect of the flange.

Examples of cases where connection flexibility may be important to model include the panel zone of steel moment-resisting frames and the "joint" region of perforated masonry or concrete walls.

C3.2.2.2 Horizontal Torsion

Research shows that effects of inelastic dynamic torsional response are more severe than effects indicated by linearly elastic models. Furthermore, it is clear that inelastic torsion can be driven both by stiffness eccentricities and by strength eccentricities; the latter are not directly indicated in linearly elastic models, but often may be identified by inspection of strengths of the earthquake-resisting components and elements. Premature failure of one or more components or elements in an otherwise symmetric structure may lead to torsional response. Structures with low levels of redundancy are likely to be more sensitive to this latter aspect than are highly redundant structures. The rehabilitation design should strive to improve the redundancy and the torsional stiffness and strength regularity of the building.

Currently, there are insufficient data available to correlate results of NSP and NDP results for torsionally sensitive systems. In the judgment of the writers, the NSP may underestimate torsional effects in some cases and overestimate effects in others.

The effects of torsion are classed as either actual, or accidental. Actual torsion is due to the eccentricity between centers of mass and stiffness. Accidental torsion is intended to cover the effects of several factors not addressed in the *Guidelines*. These factors include the rotational component of the ground motion;

differences between the computed and actual stiffnesses, strengths, and dead-load masses; and unfavorable distributions of dead- and live-load masses. The effects of accidental torsion are typically estimated by displacing the centers of mass in the same direction at one time and calculating the resulting distribution of displacements.

Checking the effects of torsion can be an onerous and time-consuming task. In the judgment of the writers, the additional effort associated with calculating the increase in component forces and deformations due to torsion is not warranted unless the effects of torsion are significant. The 10% threshold on additional displacement—due to either actual or accidental torsion—is based on judgment, not on hard data. The intent is to reward those building frames that are torsionally redundant and possess high torsional stiffness. Such structures are likely to be much less susceptible to torsional response than those framing systems possessing low redundancy and low torsional stiffness. Examples of such systems are presented in Figure C3-1.

Three-dimensional models are preferred by the writers; such models likely provide considerably improved insight into building response. However, analysis of two-dimensional mathematical models is still favored by many engineers. An increase in displacement due to torsion exceeding 50% of the displacement of the center of mass is sufficient reason to require the engineer to prepare a three-dimensional mathematical model. In the event that such increases due to torsion are calculated, the engineer is strongly encouraged to modify the layout of the framing system and to substantially increase the torsional stiffness of the building frame.

The rules presented in the *Guidelines* for including the effects of horizontal torsion for the analysis of two-dimensional models are approximate and arguably punitive. The intent of these three requirements is to provide a simple means by which to account for torsion.

Note that torsional response causes nonuniform stiffness degradation of earthquake-resisting elements, which in turn further amplifies torsion calculated from elastic analysis. This behavior is not picked up by linear procedures. Therefore, for buildings with large torsion, nonlinear procedures are recommended.

C3.2.2.3 Primary and Secondary Actions, Components, and Elements

The designation of primary and secondary actions, components, and elements has been introduced to allow some flexibility in the rehabilitation analysis and design process. Primary components, elements, or actions are those that the engineer relies on to resist the specified earthquake effects. Secondary components are those that the engineer does not rely on to resist the specified earthquake effects. Typically, the secondary designation will be used when a component, element, or action does not add considerably or reliably to the earthquake resistance. In all cases, the engineer must verify that gravity loads are sustained by the structural system, regardless of the designation of primary and secondary components, elements, and actions.

The secondary designation typically will be used when one or both of the following cases apply.

1. In the first case, the secondary designation may be used when a component, element, or action does not contribute significantly or reliably to resist earthquake effects. A gypsum partition is a component that might be designated secondary in a building because it does not provide significant stiffness or strength. A slab-column interior frame is an element that might be designated as secondary in a building braced by much stiffer and stronger perimeter frames or shear walls. Moment resistance at the pinned base of a column where it connects to the foundation is an action that might be designated as secondary because the moment resistance is low, relative to the entire system resistance.

2. In the second case, the secondary designation may be used when a component, element, or action is deformed beyond the point where it can be relied on to resist earthquake effects. An example is coupling beams connecting two wall piers. It is conceivable that these beams will exhaust their deformation capacity before the entire structural system capacity is reached. In such cases, the engineer may designate these as secondary, allowing them to be deformed beyond their useful limits, provided that damage to these secondary components does not result in loss of gravity load capacity.

The manner in which primary and secondary components are handled differs for the linear and nonlinear procedures. In the linear procedures, only primary components, elements, and actions are

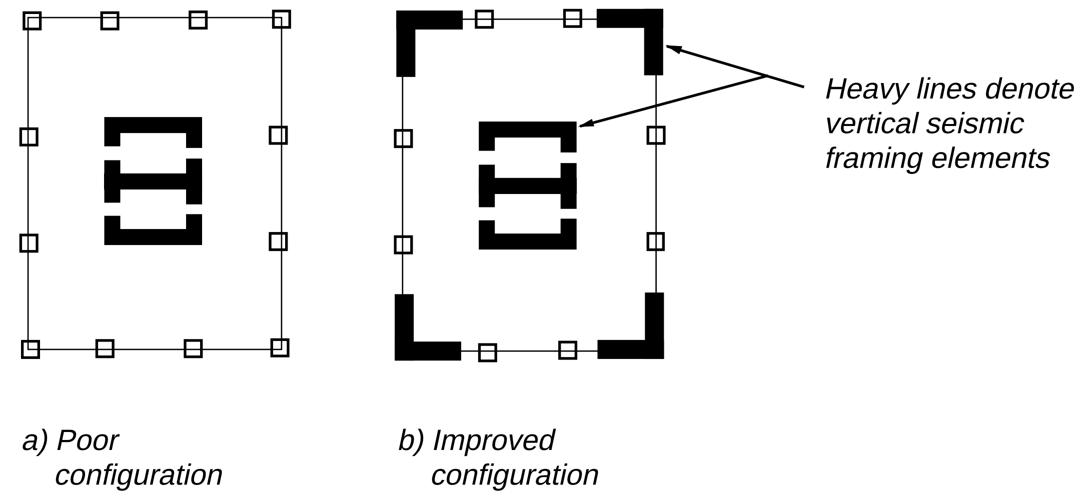


Figure C3-1 Examples of Torsional Redundancy and Torsional Stiffness

permitted to be included in the analysis model. Because of probable degradation of strength and stiffness, secondary components, elements, and actions are not permitted to be included in the linearly elastic analysis model. However, secondary components must still be checked against the acceptance criteria given in Chapters 5 through 8. In the nonlinear procedures, since strength degradation can be modeled, both primary and secondary components, elements, and actions are to be included in the nonlinear procedure model, and are to be checked against the acceptance criteria in Chapters 5 through 8.

For linear procedures, the *Guidelines* require that no more than 25% of the lateral resistance be provided by secondary components. The main reason for this limitation is that sudden loss of lateral-force-resisting components or elements can result in irregular response of a building that is difficult to detect. An example is a masonry infill wall that, if it collapses from one story of an infilled frame, may result in a severe strength and stiffness irregularity in the building. A secondary reason is to prevent the engineer from manipulating the analysis model to minimize design actions on critical components and elements. In the linear models, this 25% criterion can be checked by including the secondary components in the analysis model and examining their stiffness contribution.

Where secondary components contribute significantly to the stiffness and/or strength of the building, it is necessary to consider their effect on regularity classification of the building. In the linear procedures, it is not permitted in the analysis model to include stiffness associated with secondary components. However, if substantial secondary components result in irregular response—which can be determined by first including them in a preliminary analysis model—then the building should still be classified as irregular.

Nonstructural components and elements can profoundly, and in some cases negatively, influence the response of a building. The 10% rule of this section is based on judgment.

C3.2.2.4 Deformation- and Force-Controlled Actions

The method used for evaluating acceptance of an action is dependent on whether the action is classified as deformation-controlled or force-controlled. Deformation-controlled actions (forces or moments) are those actions for which the component has, by virtue of its detailing and configuration, capacity to deform inelastically without failure. Furthermore, a deformation-controlled action is limited to the action at the location of inelastic deformation. All other actions are designated as force-controlled actions.

Consider a cantilever column resisting axial force, shear, and bending moment. If the column has flexural ductility capacity at the connection with the footing, and if the rehabilitation design allows flexural yielding at that location, then the associated action is considered to be a deformation-controlled action. Assuming that inelastic deformation associated with axial force, shear, or moment at other locations is not permitted as part of the design, these actions are designated force-controlled actions. Table C3-1 provides examples of deformation- and force-controlled actions in common seismic framing systems.

Table C3-1 Typical Deformation-Controlled and Force-Controlled Actions

Component	Deformation-Controlled Action	Force-Controlled Action
Moment Frames • Beams • Columns • Joints	 Moment (M) M --	 Shear (V) Axial load (P), V V[1]
Shear Walls	M, V	P
Braced Frames • Braces • Beams • Columns • Shear Link	 P -- -- V	 -- P P P, M
Connections	--	P, V, M

1. Shear may be a deformation-controlled action in steel moment frame construction.

C3.2.2.5 Stiffness and Strength Assumptions

Element and component stiffness and strength assumptions specified for the *Guidelines* may differ from those commonly used in the design of new buildings. For example, reduced stiffnesses corresponding to effective cracked sections are used for concrete building analyses, whereas it has been common practice to base new designs on analyses using gross-section properties. Expected strengths, corresponding to expected material properties, are also common in the *Guidelines*, as opposed to design strengths as specified in codes for new building design. The engineer should review the stiffness and strength specifications of the relevant materials chapters of the *Guidelines* (Chapters 4 through 8, and 11) and use those values unless, through familiarity and expertise with the earthquake response and design issues, the engineer is able to identify more appropriate stiffness and strength properties.

For the NSP, it is likely that component load-deformation behavior will be represented using multilinear relationships of the types illustrated in Figure 2-4. Considerable judgment may be required in selecting the appropriate degree of complexity of the model. In most cases, simple models are preferred. The choice of the model may be guided by the following issues.

- One of the simplest component models for the NSP is a bilinear model consisting of an initial linear stiffness to yield, followed by a reduced linear stiffness. This model requires only four pieces of information: a representative elastic stiffness, the expected yield force, a post-yield stiffness, and a limiting deformation, δ_u, corresponding to a target Performance Level. Note that if a component exhibits reliable strain hardening, it is advisable to include a strain-hardening stiffness, because its neglect will lead to an overestimation of P-Δ effects and an underestimation of the maximum forces that can be delivered to force-controlled components. The bilinear model may be adequate for cases in which exceedence of the limiting deformation δ_u is unacceptable at all Performance Levels, and therefore knowledge of component behavior beyond this deformation becomes unnecessary.

- For cases in which significant component strength deterioration constitutes an acceptable state (e.g., a beam whose loss of bending resistance at the connection will not pose a life-safety hazard), the model shown as Type 1 Curve in Figure 2-4 may be appropriate. In this case, a residual strength, which could be zero, needs to be specified. The incorporation of the residual strength range in the analytical model is necessary to permit redistribution of internal forces if the deformation threshold at point 2 in the curve is exceeded.

Section 3.2.2.3 provides guidance on primary and secondary component definition, including when the stiffness of certain components, elements, or actions can be excluded from the analysis model.

C3.2.2.6 Foundation Modeling

Chapter 4 presents guidelines for stiffness and strength of foundation materials, and Chapters 5 through 8 present guidelines for steel, concrete, wood, and masonry components and elements of foundations.

Where the foundation is assumed to be rigid in the evaluation, it is necessary to evaluate the forces applied from the structure to the foundation using the acceptance criteria of Chapters 4 through 8, and 10. If the design actions exceed the allowable values, then either the structure can be rehabilitated to achieve acceptance, or the mathematical model can be modified to include the foundation according to the guidelines of Chapter 4.

C3.2.3 Configuration

Configuration plays an important role in the seismic response of buildings. Poorly-configured buildings (in many cases irregular buildings) have performed poorly in recent earthquakes (EERC, 1995; EERI, 1996). Furthermore, regular buildings can be more reliably evaluated than irregular buildings. As such, designers are encouraged to add seismic framing elements in locations that will improve the regularity of a building. Judicious location of new framing to improve regularity will simplify the analysis process and likely ensure that the analysis results will more closely represent the actual response of the building in an earthquake.

Contribution of secondary components to stiffness of the structure is expected to vary substantially during an earthquake event. In the initial earthquake excursions, secondary components are fully effective. During the latter part of an earthquake, the secondary components can lose a significant part of their strength and stiffness. For a structure to be considered regular, it needs to satisfy regularity requirements for both cases with and without contribution of secondary components.

C3.2.4 Floor Diaphragms

Floor diaphragms are a key element of the seismic load path in a building. Diaphragms transfer seismically-induced inertia forces at floor and roof levels to vertical elements of the seismic framing system, and distribute forces among vertical elements where relative stiffnesses and strengths of vertical elements differ from location to location.

In the *Guidelines*, diaphragms in provisions for Systematic Rehabilitation are classed as rigid, stiff, or flexible. Diaphragm stiffnesses in Simplified Rehabilitation are defined differently (Chapter 10). A rule for classifying diaphragm stiffness is presented; the rule is based on the relative stiffness of the diaphragm and the vertical seismic framing. Information on the stiffness and strength of diaphragms composed of different materials is presented in Chapters 5 through 8. Such information shall be used to compare the maximum lateral deformation of a diaphragm with the average inter-story drift of the story below the diaphragm.

Diaphragm flexibility results in: (1) an increase in the fundamental period of the building, (2) decoupling of the vibrational modes of the horizontal and vertical seismic framing, and (3) modification of the inertia force distribution in the plane of the diaphragm.

There are numerous single-story buildings with flexible diaphragms. For example, precast concrete tilt-up buildings with timber-sheathed diaphragms are common throughout the United States. An equation for the fundamental period of a single-story building with a flexible diaphragm is presented in Equation 3-5. Terms used in this equation are defined schematically in Figure C3-2. To calculate the fundamental period using the Rayleigh method (Clough and Penzien, 1993), a lateral load equal to the weight of the building is applied to the building in accordance with the weight distribution, and the average wall displacement, Δ_w, and diaphragm deformation, Δ_d, are calculated.

Evaluation of diaphragm demands should be based on the likely distribution of horizontal inertia forces (Mehrain and Graf, 1990). Such a distribution may be given by Equation C3-1 below; this distribution is illustrated in Figure C3-3.

$$f_d = \frac{1.5F_d}{L}\left[1-\left(\frac{2x}{L}\right)^2\right] \qquad \text{(C3-1)}$$

where:

f_d	=	Inertial load per foot
F_d	=	Total inertial load on a flexible diaphragm
x	=	Distance from the centerline of the flexible diaphragm
L_d	=	Distance between lateral support points for diaphragm

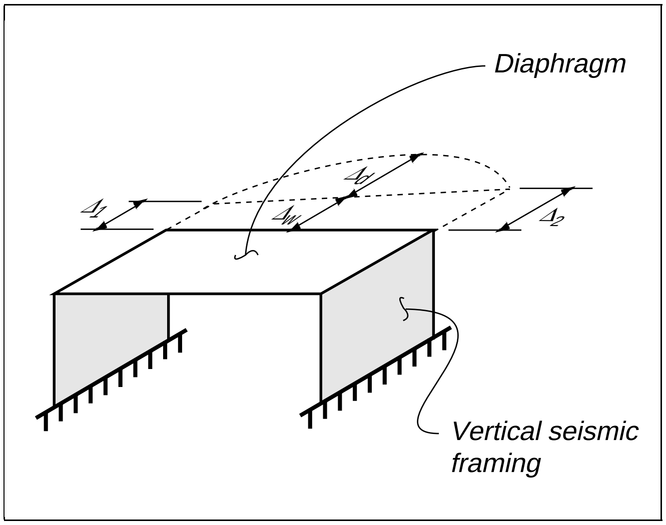


Figure C3-2 Diaphragm and Wall Displacement Terminology

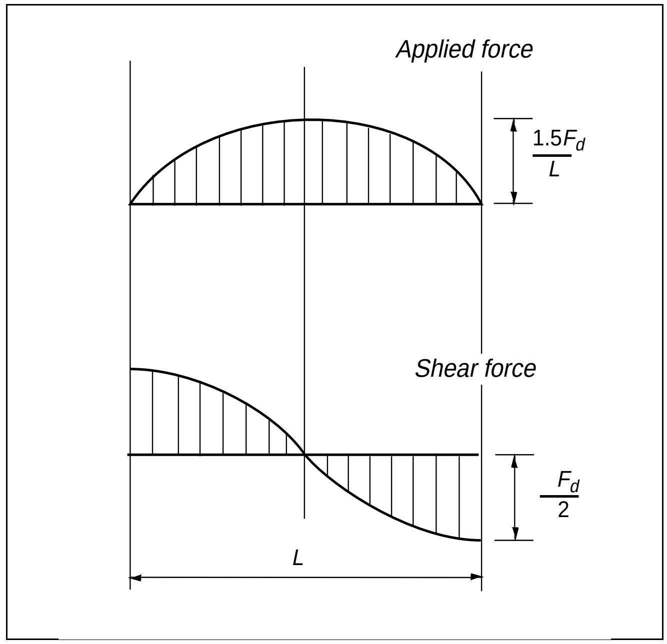
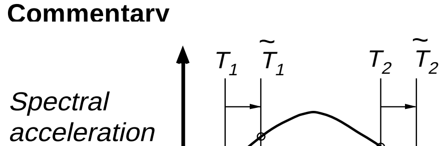


Figure C3-3 Plausible Force Distribution in a Flexible Diaphragm

C3.2.5 P-Δ Effects



element or finite difference formulation. The direct method is amenable to Linear and Nonlinear Dynamic Procedures. For the impedance function method, impedance functions representing the force-displacement characteristics of the foundation soil are specified; the soil impedance functions can be dependent or independent of excitation frequency and may include both stiffness and damping. Frequency-dependent formulations typically require frequency-domain solutions and are unsuitable for nonlinear procedures. The evaluation of foundation stiffness values, using the procedures set forth in Section 4.4.2, constitutes an impedance function approach using frequency-independent stiffness values. A discussion of methods for SSI analysis may be found in the *ASCE Standard for the Seismic Analysis of Safety-Related Nuclear Structures and Commentary* (ASCE, 1986).

C3.2.6.1 Procedures for Period and Damping

The procedures that are referenced in Section 3.2.6.1 of the *Guidelines* provide a means to calculate the effective building period and damping of the combined soil-structure system. The effective fundamental period of the building is used to determine the response spectrum acceleration used in Equation 3-6. Note that the referenced NEHRP *Provisions* (BSSC, 1995) has a typographical error in the equation for *T*. ATC (1984) Section 6.2.1 contains the correct equation for *T*.

C3.2.7 Multidirectional Excitation Effects

The rules governing multidirectional excitation effects are similar to those of BSSC (1995). Greater attention to the issue may be warranted for existing buildings, because of the greater likelihood that existing buildings will be vulnerable to brittle or low-ductility failures in force-controlled components that are overloaded by effects of multidirectional loading. The effects may be particularly important for certain vertical-load-carrying components, such as corner columns, that may receive significant overturning axial loads due to lateral loading along each of the principal horizontal axes of the building.

The 30% combination rule is a procedure that may be applied for any of the Analysis Procedures. To clarify the intention of the combination rule, consider an example of a column design. Under longitudinal loading, denote axial load as P^L, moment about *x* axis as M_x^L, and moment about *y* axis as M_y^L. Under transverse loading, similarly use P^T, M_x^T, and M_y^T. Design actions are then determined as the worse of two cases. For case one, the simultaneous design actions are axial load P, moment about *x* axis M_x, and moment about *y* axis M_y, where:

$$\begin{aligned} P &= P^T + 0.3P^L \\ M_x &= M_x^T + 0.3M_x^L \\ M_y &= M_y^T + 0.3M_y^L \end{aligned}$$

For case two, the simultaneous design actions are calculated as:

$$\begin{aligned} P &= 0.3P^T + P^L \\ M_x &= 0.3M_x^T + M_x^L \\ M_y &= 0.3M_y^T + M_y^L \end{aligned}$$

Where either the LDP or the NDP is used, the effects of multidirectional loading may be accounted for directly by applying appropriate bidirectional ground motions and directly monitoring maximum responses. Alternatively, where the LDP is used, either the 30% rule or the square root sum of squares (SRSS) rule may be used. If the objective is to find the maximum response to multicomponent ground motions for a single response quantity, a preferred approach is the SRSS combination rule. On the other hand, if the objective is to locate the response to multicomponent ground motion on a failure surface (such as a P-M_x-M_y interaction diagram for a column, as described previously), then the 30% combination rule is preferred. The complete quadratic combination (CQC) (Wilson, et al., 1981) method is not appropriate for combining actions from multidirectional ground motions.

Where the NSP is used, the 30% combination rule may be interpreted as recommending that components be checked for forces and deformations associated with the structure being displaced to 100% of the target displacement in one direction and simultaneously to 30% of the target displacement in the orthogonal direction. Limitations of currently available nonlinear analysis computer software may prevent the engineer from following this procedure explicitly. Furthermore, biaxial deformation acceptance criteria are generally lacking in Chapters 5 through 8. As an alternative, the engineer is encouraged to consider indirectly the effects of biaxial loading in implementing the evaluation. In particular, it may be important to recognize the effects of bidirectional loading on forces developed in force-controlled components. Figure C3-5 illustrates one such case, where the axial load in a corner column under bidirectional lateral loading is equal to nearly twice the axial load under unidirectional loading.

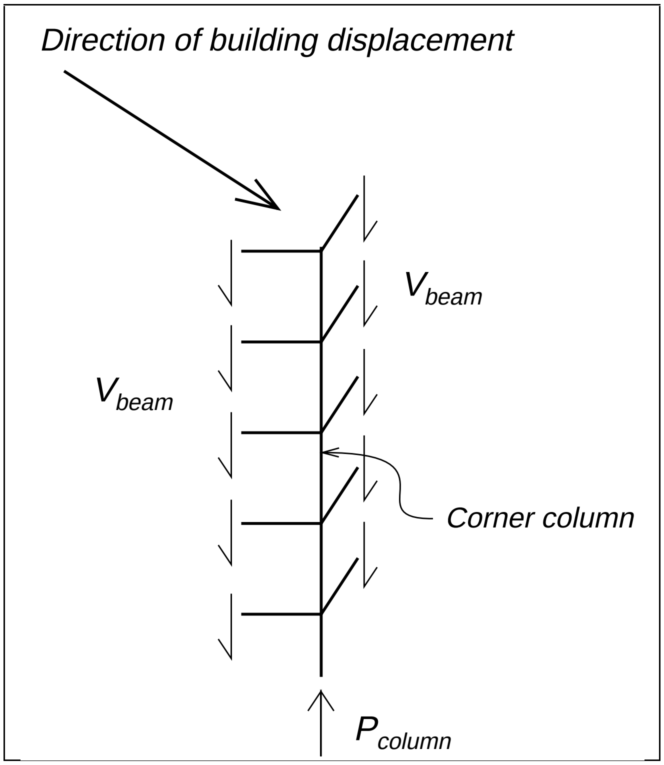


Figure C3-5 Multidirectional Effects on Calculation of Design Actions

The rule for combining multidirectional earthquake shaking effects assumes minimal correlation between ground motion components. This combination rule may be nonconservative in the near field for earthquakes with magnitudes greater than 6.5. As such, the engineer should use this rule with caution.

Vertical accelerations in past earthquakes are suspected of causing damage to long-span structures and to horizontal cantilevers. The *Guidelines* recommend that effects of vertical accelerations be considered for these structures as part of the rehabilitation design. The vertical ground shaking is defined according to Section 2.6.1.5. The procedure to be used for the analysis is the same as that described for horizontal excitations in the various portions of the *Guidelines*. Acceptance criteria are in the relevant Chapters 5 through 8. One caution with regard to vertical accelerations is that they add to gravity loads in one direction and subtract from them in the opposite direction. The possibility that response will be skewed in one direction or the other, and that plastic deformations may accumulate in the direction of gravity loads, should be considered.

C3.2.8 Component Gravity Loads and Load Combinations

In general, both the load combinations represented by Equations 3-2 and 3-3 should be analyzed as part of the Systematic Rehabilitation Method. For the linear procedures, superposition principles can be used to develop design actions for the different load cases—a relatively simple process involving algebraic manipulation of results obtained from lateral and gravity load analyses. For the nonlinear procedures, superposition cannot, in general, be used, so that application of both Equations 3-2 and 3-3 requires two completely separate analyses, a process that may require considerable effort. It may be possible in certain cases to determine by inspection that one of the two gravity load combinations will not control the design.

The load case represented by Equation 3-3 is critical for cases where earthquake effects result in actions that are opposite those due to gravity loads. Although these cases are seemingly ubiquitous and noncritical in any structure, they are considered especially critical for force-controlled components or actions. Examples include tension forces in corner columns and in vertical chords of shear walls and braced frames.

The gravity load combinations set forth in Equations 3-2 and 3-3 for use in seismic evaluation differ from those presented in regulations for new construction. The resulting member actions are smaller than those calculated for corresponding new construction. The gravity load combinations were modified on the following bases: (1) the *Guidelines* require on-site evaluation of dead loads and permanent live loads, thereby reducing the likely scatter in the magnitudes of the gravity loads assumed for analysis; (2) the building is known to have existed under the action of loads and is known to be adequate for those loads; (3) the Performance Levels identified in the *Guidelines* are not necessarily the same as those implicit in the design basis for new buildings; and (4) the *Guidelines* use different definitions of materials and component strengths from those used for the design of new buildings.

The component loads and load combinations presented in Equations 3-2 and 3-3 are intended for seismic evaluation only. Component loads and load combinations for gravity and wind load checking are identified in other regulations; the component loads and load combinations set forth in Section 3.2.8 must not be used for gravity load evaluation.

The minimum live load specification equal to 0.25 of the unreduced design live load is a traditionally applied value used in design to represent the likely live load acting in a structure. Where the load is likely to be larger, use this larger load.

C3.2.9 Verification of Design Assumptions

The goals of this section are (1) to require the engineer to check design actions and associated strengths at all locations within the component rather than just at the end points or nodes used to define the component in the mathematical model, and (2) to ensure that the post-earthquake residual gravity-load capacity of a component is not substantially compromised due to redistribution of moments resulting from earthquake shaking. The first goal addresses component response during earthquake shaking; the second addresses component response following earthquake shaking. High gravity-load actions, identified using the 50% rule presented in the *Guidelines*, will increase the likelihood that these items will be critical for design.

If component actions due to gravity loads are much smaller than the expected component strengths at all locations, it is neither probable that flexural hinges will form between the component ends nor is it likely that flexural hinges will form between the component ends due to small increases in gravity loads following an earthquake. The 50% rule presented in the *Guidelines* is based on the judgment of the writers. Note that this comparison of component actions and strengths is based on the load combinations set forth in Equations 3-2 and 3-3, and not on load combinations set forth in other regulations for gravity load checking. For components with gravity load actions exceeding the 50% rule, verification of Item 1 is mandatory, and checking for Item 2 is recommended.

Hinge Formation at Component Ends. For beams evaluated or designed using the linear procedures, inelastic flexural action normally should be restricted to the beam ends. This is because linear procedures can lead to nonconservative results, and may completely misrepresent actual behavior, when flexural yielding occurs along the span length (that is, between the component ends). To check for flexural yielding along the span length, construct a free-body diagram of the beam loaded at its ends with the expected moment strengths Q_{CE} and along its length with the gravity loads given by Equations 3-2 and 3-3. (See Figure C3-6 for details.) The moment diagram can then be constructed from equilibrium principles. The moments along the length of the beam can then be compared with the strengths at all locations. For this purpose, the strength may be calculated as an expected strength rather than a lower-bound strength. Where this comparison indicates that flexural strength may be reached at locations more than one beam depth from the beam ends, either the beam should be rehabilitated to prevent inelastic action along the length, or the design should be based on one of the nonlinear procedures (Sections 3.3.3 or 3.3.4).

For beams evaluated or designed using the nonlinear procedures, it is required that inelastic flexural actions be restricted to nodes that define the beam in the mathematical model. It is recommended that nodes be placed at the locations of significant mass and/or reactions (likely corresponding to the locations of maximum gravity moments). To check for flexural yielding along the span length, construct a free-body diagram of the beam loaded at its ends with the moments calculated by nonlinear procedures and along its length with the gravity loads given by Equations 3-2 and 3-3. This is similar to that shown in Figure C3-6, except that calculated moments from nonlinear procedures replace the expected strengths calculated by cross-section analysis. The moment diagram can then be constructed from equilibrium principles. The moments along the length of the beam can then be compared with the strengths at all locations. For this purpose, the strength may be calculated as an expected strength rather than a lower-bound strength. Where this comparison indicates that flexural strength may be reached at locations other than nodes in the mathematical model, the mathematical model should be refined and the building reanalyzed.

Post-Earthquake Residual Gravity Load Capacity. Earthquake shaking can substantially affect the magnitude of gravity load actions in a building frame. Consider a steel beam in a simple building frame shown in Figure C3-7. Assume that the beam moment strength is constant along its length. The gravity moment diagram is shown in Figure C3-7a. At the beam ends the gravity moment is equal to 50% of the beam strength, while at the mid-span of the beam the gravity moment is equal to 75% of the beam strength. (For this beam the total static moment due gravity loads is equal to 125% of the beam strength.) Evaluation of this beam for gravity moment strength would find this beam adequate at all locations. Due to moment redistribution within the frame, it is plausible that the post-earthquake moment diagram due to gravity loads could be that given by Figure C3-7b. At the beam ends the gravity moment is equal to 25% of the beam strength. At the mid-span of the beam the gravity moment is equal to 100% of the beam strength. Although evaluation of this beam for gravity moment strength would find this beam adequate at all locations, any increase in gravity loads would produce flexural hinging at the mid-span of the beam. If this beam is not designed for ductile behavior at this location, local failure of the beam may ensue. (Note that the moment diagrams presented in Figures C3-6 and C3-7 are somewhat arbitrary, and are intended to illustrate the issues identified above.)

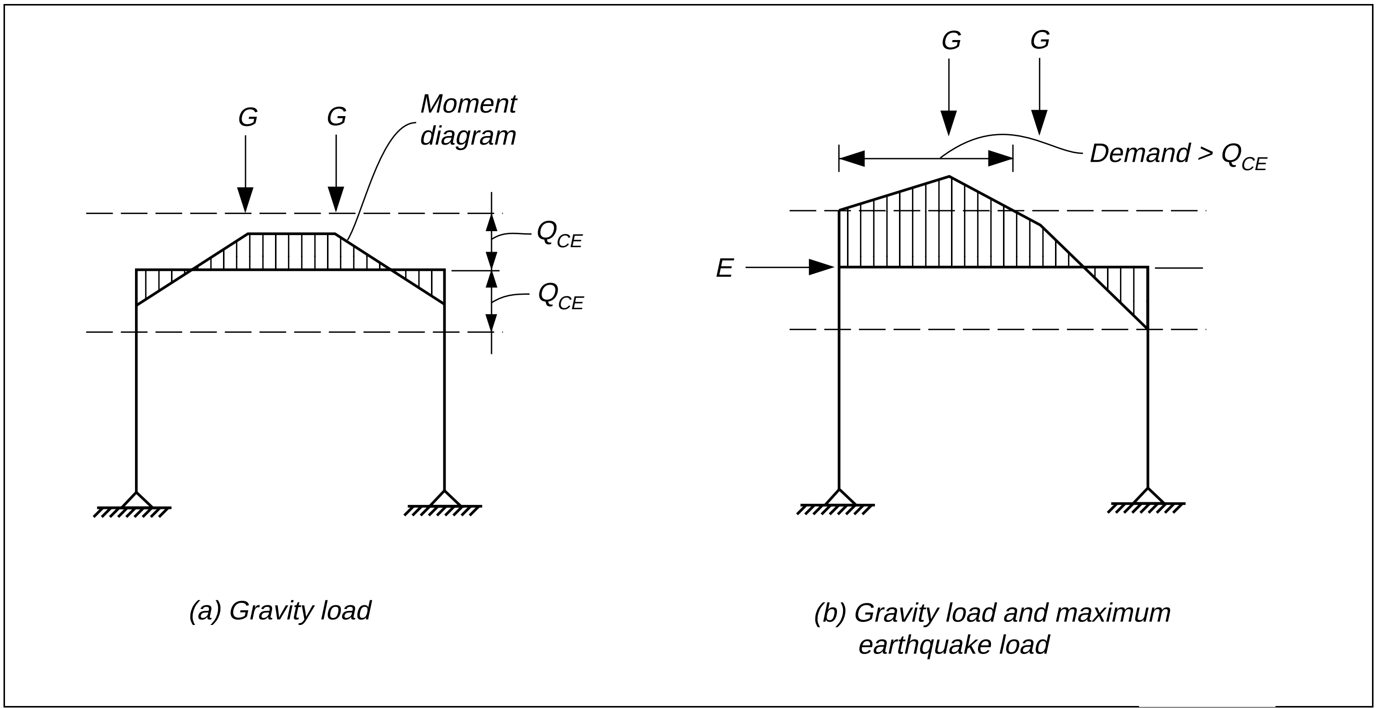


Figure C3-6 Hinge Formation Along Beam Span

For beams designed using linear procedures, a very conservative method for checking post-earthquake residual gravity-load capacity is to load the beam ends with zero moment and the beam along its length with the gravity loads given by Equation 3-2 or 3-3.

For beams designed using the NSP, one method for checking post-earthquake residual gravity-load capacity is to unload the frame (that is, load the frame with lateral forces equal and opposite to those corresponding to the target displacement, for a total of zero applied lateral load). Gravity loads should be applied through all stages of the analysis. For beams designed using the

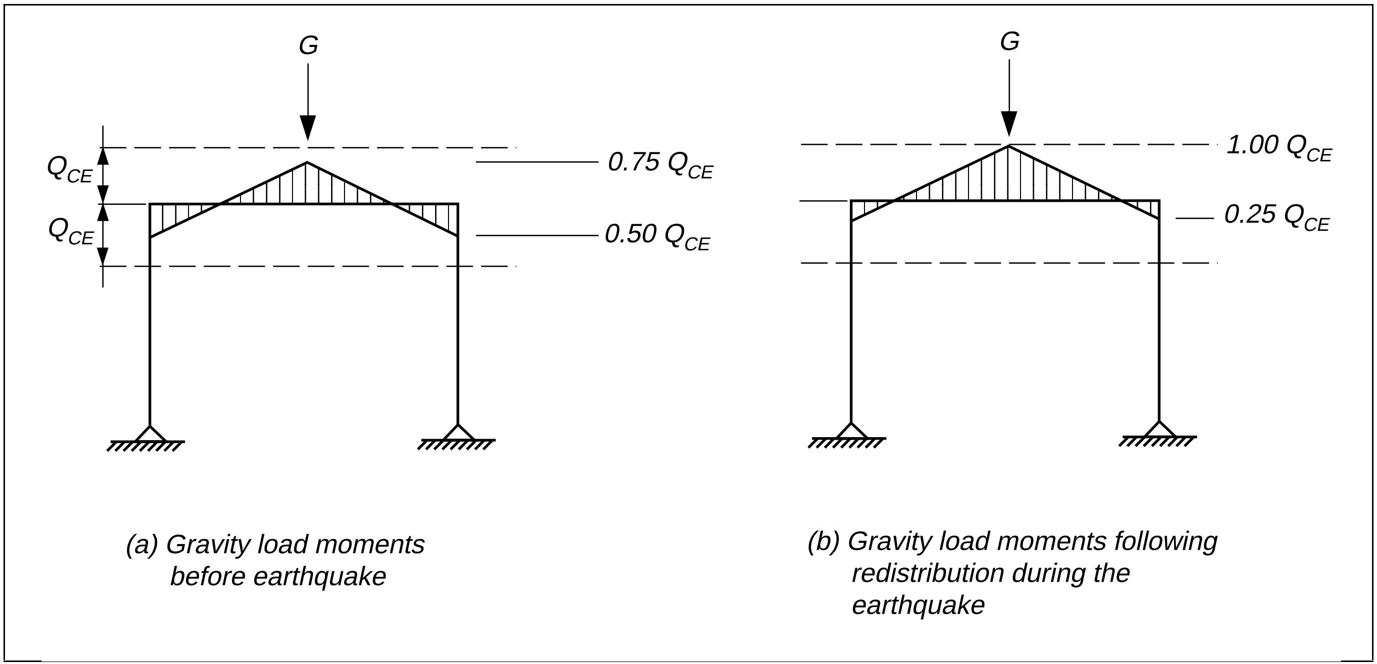


Figure C3-7 Post-Earthquake Residual Gravity Load Capacity

NDP, the effects of moment redistribution due to earthquake shaking can be directly evaluated by review of the gravity load actions at the end of the time-history analysis.

Rules for minimum residual gravity load capacity above that required by the load combinations set forth in Equations 3-2 and 3-3 are not provided because the residual capacity is likely a function of the Performance Level used for the design. The engineer should develop rules on a project-by-project basis. The reader is referred to Bertero (1996) for additional information.

C3.3 Analysis Procedures

The *Guidelines* present four specific Analysis Procedures. The writers recognize that variations on these procedures—and completely different procedures—are currently in use, and that these alternate procedures may be equally valid, and in some cases may provide added insight into the evaluation and design process. Some of these alternative procedures, described in this *Commentary*, may be considered to be acceptable alternatives to the four procedures presented in the *Guidelines*, although the engineer should verify that they are applicable to the particular conditions of the building and its Rehabilitation Objectives.

C3.3.1 Linear Static Procedure (LSP)

C3.3.1.1 Basis of the Procedure

According to the LSP, static lateral forces are applied to the structure to obtain design displacements and forces. Two important assumptions are involved. First, it is implied that an adequate measure of the design actions can be obtained using a static analysis, even though it is recognized that earthquake response is dynamic. Section 2.9 provides criteria to determine when this simplification is unsatisfactory, and when dynamic analysis is required as an alternative. Second, it is implied that an adequate measure of the design actions can be obtained using a linearly-elastic model, even though nonlinear response to strong ground shaking may be anticipated. Section 2.9 provides criteria to determine when this assumption is unsatisfactory, and when nonlinear procedures are required as an alternative. In general, the writers of the *Guidelines* recognize that improved estimates of response quantities can be obtained using dynamic analysis, and further improvements can be obtained using nonlinear response analysis where nonlinear response is anticipated. Use of these approaches is encouraged.

The *Guidelines* adopt a widely-accepted philosophy that permits nonlinear response of a building when subjected to a ground motion that is representative of the design earthquake loading. For some structures, total allowable deformations may be several times higher than yield deformations. The primary measure of the performance of a "yielding" building lies in the level of deformation imposed on individual components and elements, compared with their reliable deformation capacities. Stress, force, and moment amplitudes are of secondary importance for ductile components and elements, as it is accepted that ductile materials will reach their stress capacities, and be deformed beyond the yield point. Stress, force, and moment amplitudes may be of primary importance for brittle (force-controlled) components and elements that may fail when force demands reach force capacities.

Ideally, the evaluation of a "yielding" building should be carried out using nonlinear procedures that explicitly account for nonlinear deformations in yielding components. As an alternative, the *Guidelines* permit evaluation to be carried out using linear procedures. In a linear procedure, there is a direct relation between internal forces and internal deformations for any given loading pattern. Therefore, it is simpler when using linear procedures to express acceptability in terms of internal forces rather than internal deformations. This is the approach adopted with the LSP.

Figure C3-8 illustrates the intent of the LSP. The solid curve in the figure represents the backbone load-displacement relation of the building as it is deformed to the maximum displacement δ_{max} by the design earthquake loading. The LSP represents the building by a linearly-elastic stiffness that approximately corresponds to the effective lateral load stiffness for loading below the effective yield point of the building. To achieve the maximum displacement, δ_{max}, using the linearly-elastic model, the model must be loaded by a pseudo lateral load *V* defined by Equation 3-6. This pseudo lateral load may be several times larger than the base shear capacity of the building, and corresponding internal component forces may similarly be several times the component force capacities. The acceptance procedures of Section 3.4 take this aspect into account, allowing component overstress levels that vary with the expected nonlinear deformation capacity of the individual component.

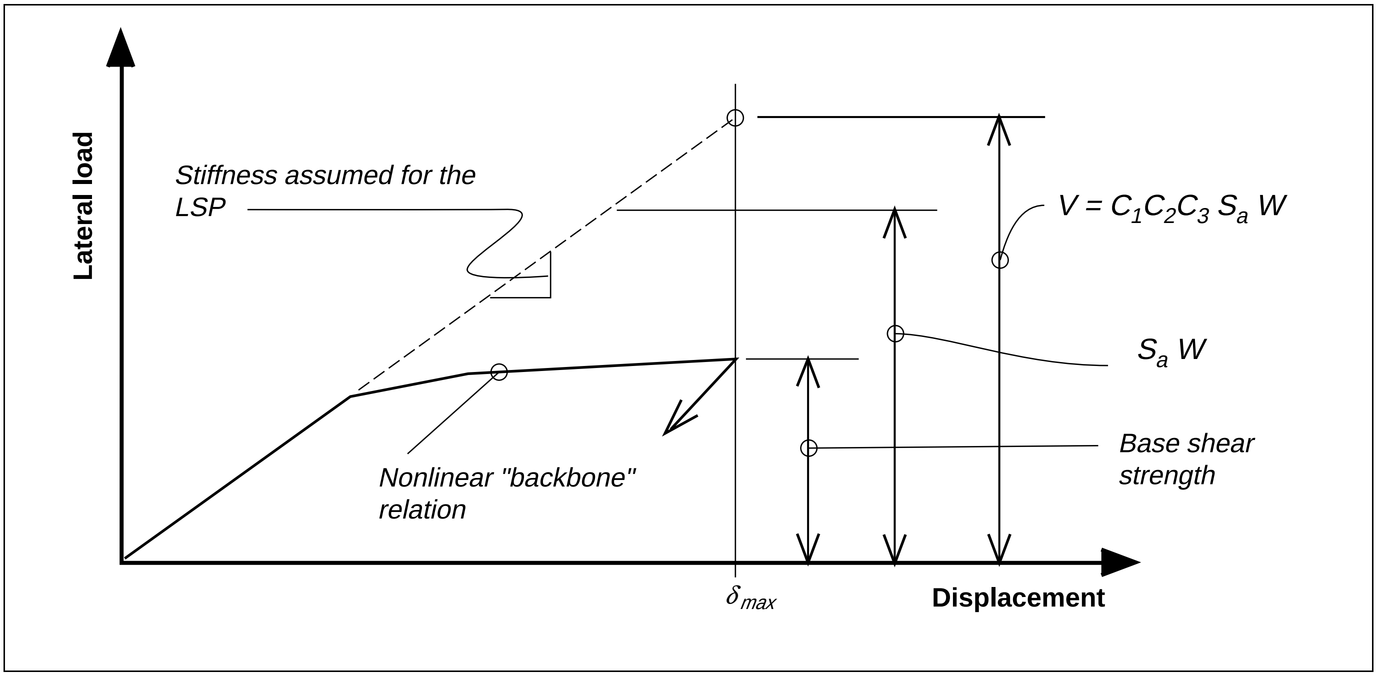


Figure C3-8 Basis for the Linear Static Procedure

C3.3.1.2 Modeling and Analysis Considerations

The following commentary contains essential details of the LSP.

A. Period Determination

In accordance with the basis of the LSP as illustrated in Figure C3-8, the period used for design should correspond to the fundamental translational period of the building responding in the linearly-elastic range. Other definitions of period—for example, secant values—are not generally appropriate for the LSP.

For many buildings, including multistory buildings with well-defined framing systems, the preferred approach to obtaining the period for design is Method 1. By this method, the building is modeled using the modeling procedures of Chapters 4 through 8, and 11, and the period is obtained by eigenvalue analysis. Flexible diaphragms may be modeled as a series of lumped masses and diaphragm finite elements. Many programs available from commercial software providers are capable of determining the period specified in Method 1.

Method 2 provides an approximate value of the fundamental translational period for use in design. The expressions for period are the same as those that appear in the *NEHRP Recommended Provisions for Seismic Regulations for New Buildings* (BSSC, 1995). Method 2 may be most suitable for small buildings for which detailed mathematical models are not developed. Method 2 may also be useful to check that periods calculated by Method 1 are reasonable. On average, actual measured periods, and those calculated according to Method 1, exceed those obtained by Method 2.

Method 3 applies only to one-story buildings with single span flexible diaphragms. Equation 3-5 is derived from an assumed first-mode shape for the building (Figure C3-2). The equation is not applicable to other buildings.

Periods obtained from the three different methods should not be expected to be the same, as each is based on a different set of approximating assumptions. Design forces and displacements in the LSP are intended to be obtained by applying a pseudo lateral load (Section 3.3.1.3A) to a mathematical model of the building. The most conservative design results will be obtained for the period that produces the maximum pseudo lateral load. Usually, this will be achieved by using a low estimate of the fundamental period, although for certain site-specific spectra the opposite will be the case. The engineer should investigate this possibility on a case-by-case basis.

The approximate formula, $T = 0.1N$, for the period T of steel or reinforced concrete moment frames of 12 stories or less ($N \le 12$) is added here for historical completeness.

C3.3.1.3 Determination of Actions and Deformations

A. Pseudo Lateral Load

The pseudo lateral load is the sum of lateral inertial forces that must be applied to the linearly-elastic model of the building to produce displacements approximately equal to those the actual structure is expected to undergo during ground motion corresponding to the design earthquake loading. In Equation 3-6, the quantity $S_a W$ is the elastic spectral force associated with the design earthquake loading. When this force is applied to a linearly-elastic model of the structure, it produces deformations expected for the linearly-elastic structure subjected to the design earthquake loading. Coefficients C_1, C_2, and C_3 modify the elastic force levels for the purpose of correspondingly modifying the design deformations in the “yielding” structure. The effect of coefficient C_1 is illustrated in Figure C3-9. Note that the purpose of the coefficients is to modify the design displacements to be more representative of those expected for a “yielding” s$T = 0.1N$ ubjected to the design earthquake loading.

The anticipated live load in W is different from the Q_L of Section 3.2.8.

Note that reduction of base shear due to multimode effects has conservatively not been used in the LSP.

Further discussion on the coefficients in Equation 3-6 follows.

Coefficient C_1. This modification factor is to account for the difference in maximum elastic and inelastic displacement amplitudes in structures with relatively stable and full hysteretic loops. The values of the coefficient are based on analytical and experimental investigations of the earthquake response of yielding structures (Nassar and Krawinkler, 1991; Miranda and Bertero, 1994; Bonacci, 1989). The continuous curves in Figure C3-9 illustrate mean values of the coefficient C_1 as formulated by Miranda and Bertero (1994). In that figure, the quantity R is the ratio of the required elastic strength to the yielding strength of the structure.

Where the quantity R is defined, it is preferable to use the appropriate values of C_1 given by the continuous curves in Figure C3-9. Where the quantity R is not defined, as permitted for the LSP, the coefficient C_1 may be read from the broken curve in Figure C3-9, which is a graphical representation of the expressions given in Section 3.3.1.3A.

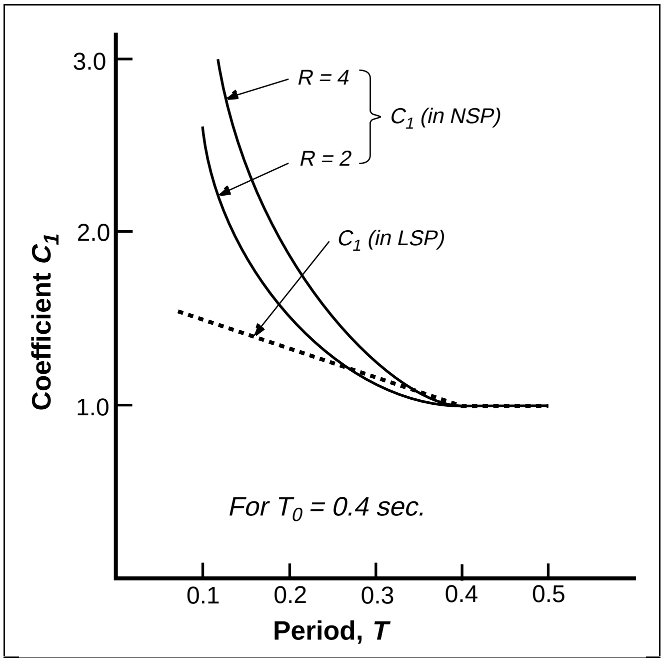


Figure C3-9 Relation between R and C_1

Note that the relations represented in Figure C3-9 are mean relations, and that considerable scatter exists about the mean (Miranda, 1991). For critical structures, the engineer should consider increasing the value of the coefficient C_1 to account approximately for the expected scatter.

Recent studies by Constantinou et al. (1996) suggest that maximum elastic and inelastic displacement amplitudes may differ considerably if either the strength ratio R is large or the building is located in the near-field of the causative fault. Specifically, the inelastic displacements will exceed the elastic displacement. If the strength ratio exceeds five, it is recommended that a displacement larger than the elastic displacement be used as the basis for calculating the target displacement.

Coefficient C_2. The above description of Coefficient C_1 is based on mean responses of inelastic single-degree-of-freedom (SDOF) systems with bilinear hysteresis models. If the hysteresis loops exhibit significant pinching or stiffness deterioration (Figure C6-22), the energy absorption and dissipation capacities decrease, and larger displacement excursions should be expected. At the time of this writing, only limited data are available to quantify this increase in displacement, but it is known that this effect is important for short-period, low-strength structures with very pinched hysteresis loops. Pinching is a manifestation of structural damage; the smaller the degree of nonlinear response, the smaller the degree of pinching. Framing Types 1 and 2 are introduced for the purpose of cataloguing systems prone to exhibit pinching and strength degradation—that is, Type 1. Type 2 systems are those not specifically identified as Type 1. Values for C_2 are reduced for smaller levels of damage; that is, the values for C_2 are smaller for Immediate Occupancy (little-to-no damage) than for Collapse Prevention (moderate-to-major damage). The period-dependence of this displacement modifier has been established by analysis; sample data comparing the displacement responses of a severely pinched SDOF system and a bilinear SDOF system are presented in Figure C3-10 (Krawinkler, 1994).

Framing systems whose components exhibit pinched hysteresis will likely experience strength degradation in severe earthquakes. This deterioration will further increase earthquake displacements. The values for C_2 given in Table 3-1 are intended to account for both stiffness degradation and strength deterioration, and are based on judgment at the time of this writing.

Coefficient C_3. For framing systems that exhibit negative post-yield stiffness, dynamic P-Δ effects may lead to significant amplification of displacements. Such effects cannot be explicitly addressed with linear procedures. The equation given for coefficient C_3 for flexible buildings ($\theta > 0.1$), namely:

$$C_3 = 1 + \frac{5(\theta - 0.1)}{T} \tag{C3-2}$$

is loosely based on the equation for coefficient C_3 presented for use with the NSP. Note that no measure of

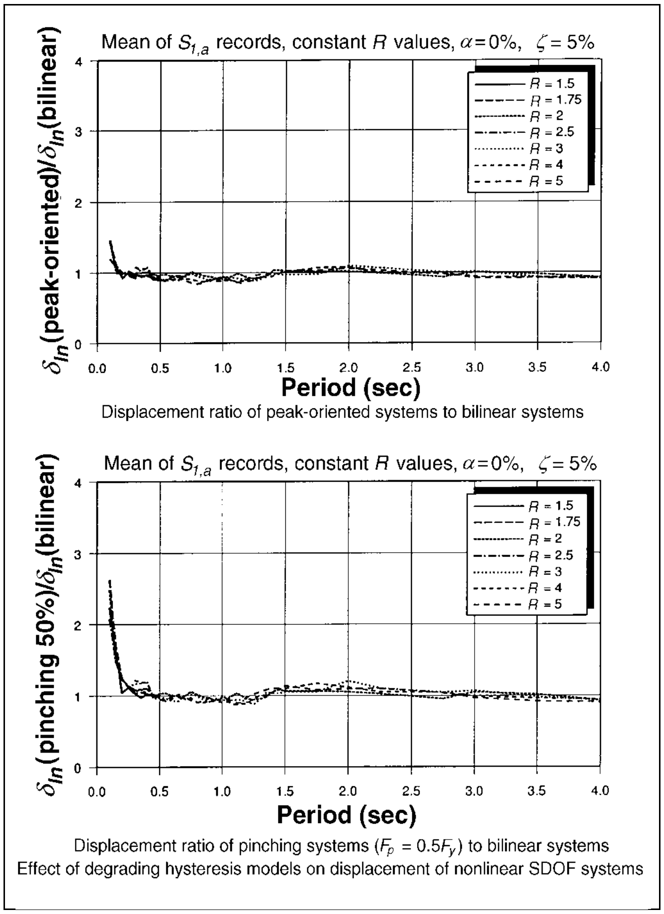


Figure C3-10 Increased Displacements Due to Pinched Hysteresis

the degree of negative post-yield stiffness can be explicitly included in a linear procedure.

B. Vertical Distribution of Seismic Forces

The distribution of inertia forces over the height of a building during earthquake shaking varies continuously in a complex manner. Sample inertia force distributions are presented in Figure C3-11. Key to design is capturing the critical distribution(s) that will maximize design actions.

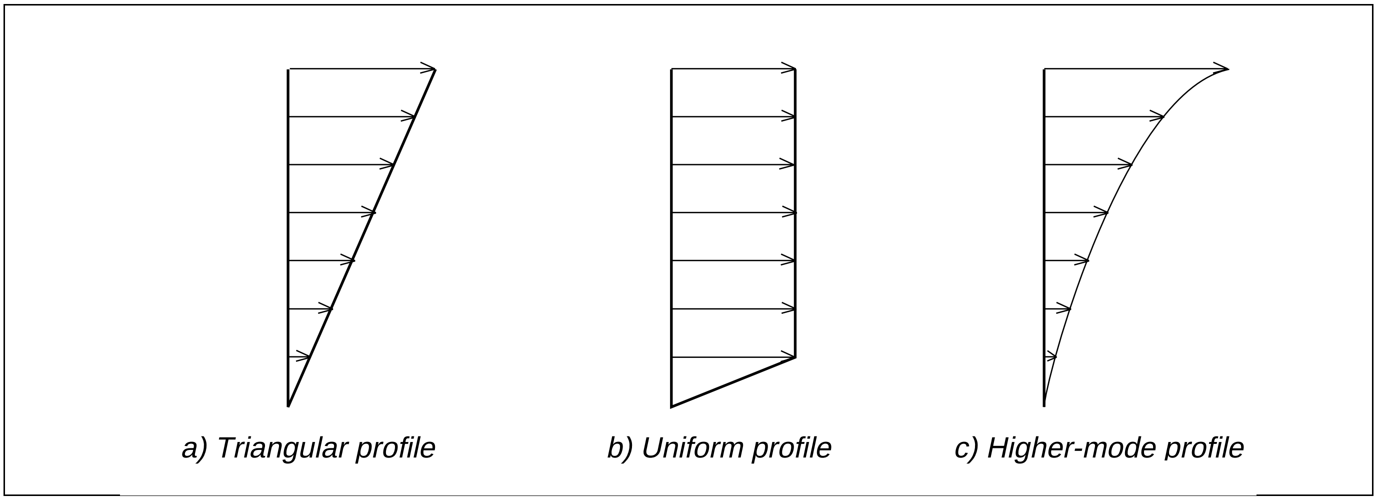


Figure C3-11 Sample Inertia Force Distributions

If the building is responding in the linearly-elastic range, the distribution of inertia forces is a function of many factors, such as the frequency characteristics and amplitude of the earthquake shaking, and the modal frequencies and shapes of the building. If the building is responding in the nonlinear (inelastic) range, the distribution of inertia forces is further complicated by localized, and perhaps global, yielding in the building.

For analysis and design, simplified procedures are needed that will likely capture the worst-case distribution of inertia forces. The method for vertical distribution of seismic forces assumes linear response in the building; the method is virtually identical to that used in the *NEHRP Recommended Provisions for Seismic Regulations for New Buildings* (BSSC, 1995).

For short-period buildings ($T \le 0.5$ second), the vertical distribution of inertia forces assumes first-mode response only — approximated by setting k equal to 1.0. The resulting inertia force distribution is the inverted-triangular distribution that formed the basis of seismic design provisions for many years.

For long-period buildings ($T \ge 2.5$ seconds), higher-mode effects may substantially influence the distribution of inertia forces, producing higher relative accelerations in the upper levels of a building. Higher mode effects are introduced using a value of k greater than 1.0. The use of values of k greater than 1.0 has the effect of increasing both the story shear forces in the upper levels of a building, and the global overturning moment for a given base shear, by moving the seismic force resultant up toward the roof of the building. Note that increasing the ratio of moment to shear demand may not be conservative in the design of shear-critical elements such as reinforced-concrete structural walls.

C. Horizontal Distribution of Seismic Forces

The inertia forces F_x from Equation 3-7 arise from acceleration of the individual masses attributed to floor level x. Therefore, this section specifies that the forces F_x be distributed across the level in proportion to the mass distribution of the floor.

The total story shear force, overturning moment, and horizontal torsional moment are to be determined from statics considering the application of the inertia forces to the levels above the story being considered. The distribution of these to individual resisting elements is to be determined by analysis, considering equilibrium and compatibility among the vertical and horizontal elements of the structural system.

D. Floor Diaphragms

The floor diaphragm is a key component of the seismic load path in a building. Diaphragms serve to transfer seismic-induced inertia forces to vertical members of the seismic framing system.

The connection between a diaphragm and the associated vertical seismic framing element is a critical element in the seismic load path. Buildings have failed during earthquake shaking due to a lack of strength in such connections. Diaphragm connections should be designed to have sufficient strength to transfer the maximum calculated diaphragm forces to the vertical framing elements.

The seismic loading in the plane of a diaphragm includes the distributed inertia force equal to the response acceleration at the level of the diaphragm multiplied by its distributed mass. Equation 3-9 provides an approximate method for determining the seismic forces for design. Coefficients C_1, C_2, and C_3 are removed from the diaphragm inertia force calculation because they are displacement multipliers (on vertical lateral-force-resisting elements) and not force multipliers. The diaphragm must also be designed to transfer the concentrated shear forces from vertical seismic framing above the diaphragm to vertical seismic framing below the diaphragm wherever there are changes in the stiffness or plan location of such framing.

C3.3.2 Linear Dynamic Procedure (LDP)

C3.3.2.1 Basis of the Procedure

The LDP uses the same linearly-elastic structural model as does the LSP. Because the LDP represents dynamic response characteristics directly, it may provide greater insight into structural response than does the Linear Static Procedure. However, as with the LSP, it does not explicitly account for effects of nonlinear response. The writers of the *Guidelines* recognize that improved estimates of response for use in design may be achieved in many cases by using nonlinear response analysis, and encourage the use of the nonlinear procedures where appropriate.

Section C3.3.1.1 provides additional discussion of the basis of the linear procedures.

C3.3.2.2 Modeling and Analysis Considerations

A. General

For the LDP, the results of linear dynamic analysis are not scaled to the base shear from the LSP. Thus, the equivalent base shear in the LDP is expected to be lower than the value obtained from the LSP, due to higher-mode participation effects.

B. Ground-Motion Characterization

The Response Spectrum Method uses either the response spectrum as defined in Section 2.6.1.5 or a site-specific response spectrum as defined in Section 2.6.2.1. The Time-History Method uses ground-motion time histories as defined in Section 2.6.2.2.

C. Response Spectrum Method

The Response Spectrum Method requires dynamic analysis of a mathematical model of a building to establish modal frequencies and mode shapes. Using standard mathematical procedures (Clough and Penzien, 1993) and a response spectrum corresponding to the damping in the building, the modal frequencies and shapes are used to establish spectral demands. The spectral demands are then used to calculate member forces, displacements, story forces, story shears, and base reactions for each mode of response considered. These forces and displacements are then combined using an established rule to calculate total response quantities.

The *Guidelines* require that a sufficient number of modes of response be considered in the analysis so as to capture at least 90% of the building mass in each of the building's principal horizontal directions. The 90% rule is the industry standard and has been used in the *NEHRP Recommended Provisions for Seismic Regulations for New Buildings* and the *Uniform Building Code* for many years.

Two modal combination rules are identified in the *Guidelines*. The first, the square root sum of squares (SRSS) rule (Clough and Penzien, 1993), has been widely used for more than 30 years. The second, the complete quadratic combination (CQC) rule (Wilson et al., 1981) has seen much use since the mid-1980s. The reader is referred to the literature for additional information.

Requirements for simultaneous, multidirectional seismic excitation are given in Section 3.2.7.

D. Time-History Method

The Time-History Method involves a step-by-step analysis of the mathematical model of a building using discretized earthquake time histories as base motion inputs. Torsional effects shall be captured explicitly using the Time-History Method. Time-History Analysis of two- and three-dimensional mathematical models is permitted by the *Guidelines*. Three-dimensional mathematical models may be analyzed using either ground-motion time histories applied independently along each principal horizontal axis, or orthogonal ground-motion time histories (constituting a pair of time histories) applied simultaneously.

Earthquake ground-motion time histories, and pairs of such time histories, shall be established in accordance with the requirements of Section 2.6.2.2. Correlation between ground-motion time histories that constitute a pair of ground-motion time histories shall be consistent with the source mechanism and assumed epicentral distance to the building site.

Multidirectional excitation effects can be considered by either (1) simultaneously applying pairs of ground-motion time histories to the mathematical model (with appropriate phasing of the ground motion components), or (2) following the procedures set forth in Section 3.2.7.

C3.3.2.3 Determination of Actions and Deformations

A. Modification of Demands

The actions and deformations calculated using either the Response Spectrum or Time-History Methods shall be factored by the coefficients C_1, C_2, and C_3 developed for the LSP. For information on these coefficients, the reader is referred to the commentary above.

B. Floor Diaphragms

The reader is referred to the commentary on Section 3.3.1.3D for pertinent information. The 85% rule of Section 3.3.2.3B is intended to offer the engineer an incentive to use the LDP; the value of 85% is arbitrary.

C3.3.3 Nonlinear Static Procedure (NSP)

C3.3.3.1 Basis of the Procedure

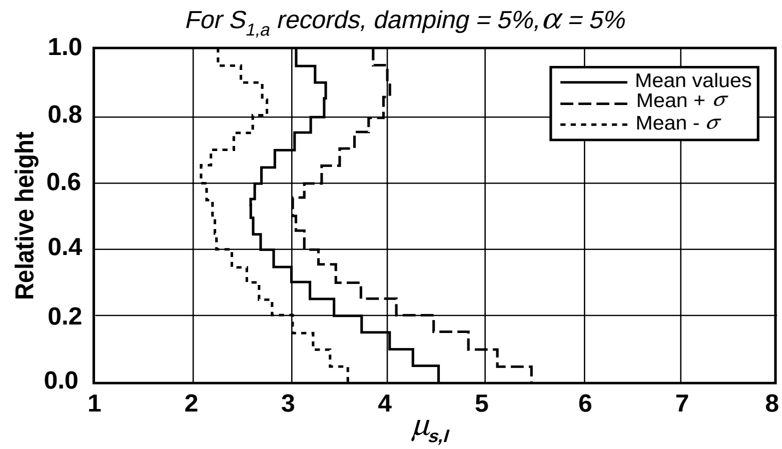
According to the NSP, static lateral forces are applied incrementally to a mathematical model of the structure until a target displacement is exceeded. Building deformations and internal forces are monitored continuously as the model is displaced laterally. The procedure parallels that of the LSP, but with two very important differences. First, in the NSP the nonlinear load-deformation behavior of individual components and elements is modeled directly in the mathematical model. Second, in the NSP the earthquake effect is defined in terms of a target displacement rather than a pseudo lateral load. The NSP requires that the behavior of components in which internal forces reach strengths be described by multilinear (in the simplest case, bilinear) force-deformation models with well-defined strength and deformation capacities. The design force and deformation demands in each component are calculated for the design earthquake displacement(s), and acceptability is evaluated by comparing the computed force and deformation demands with available capacities. Capacities for different Performance Levels are provided in Chapters 4 through 9, and 11. Although the NSP requires considerably more analysis effort than does the LSP, it usually provides improved insight into the expected nonlinear behavior of the structure, and therefore usually provides better design information.

The NSP uses ground motion information derived from smoothed design spectra, thereby avoiding the narrow valleys and peaks that often characterize individual ground motion records, and consequently providing a more robust design loading. The procedure’s shortcoming is its inability to represent realistically all changes in nonlinear dynamic response characteristics of the structure caused by cyclic stiffness degradation and strength redistribution. This shortcoming may lead to deficient estimates of local force and plastic deformation demands, particularly when higher modes gain in importance as yielding progresses in the structure. Thus, when higher modes are important, preference should be given to the NDP. Chapter 2 presents restrictions on the use of the NSP based on considerations of the higher-mode dynamic effects.

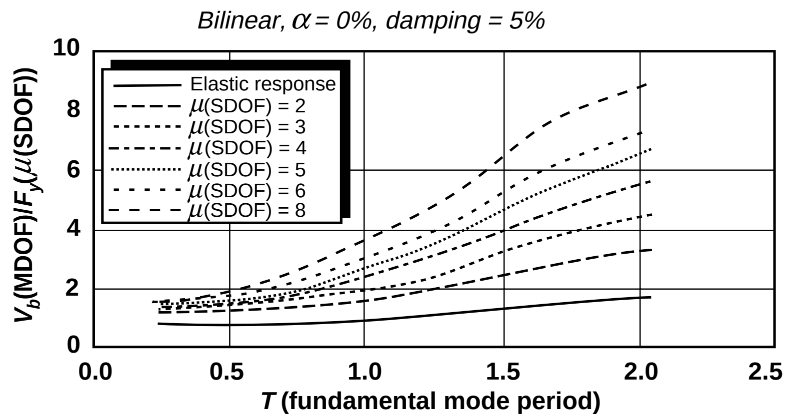
It is possible, when evaluating a building having multiple failure modes, that the NSP will identify only one of these modes, effectively overlooking the other modes. An example is a multistory building with weak columns in multiple floors. Analysis by the NSP using a single lateral load distribution is likely to identify vulnerability of only a single floor, especially if there is insignificant strain-hardening associated with column failure. The other floors may be equally or more vulnerable to collapse under dynamic loading for which the lateral inertia force distribution is continually changing. The NSP requires that at least two lateral load distributions be considered in the evaluation, in part to identify the potential for multiple failure modes. The engineer needs to be generally aware that multiple failure modes may be possible, and needs to implement rehabilitation strategies that mitigate the vulnerabilities in each of these modes.

Figure C3-12 illustrates some of the limitations of the NSP. The top diagram shows the mean and mean $\pm\ \sigma$ values of the story ductility demands for a 1.2-second frame structure subjected to a set of 15 ground motion records (Seneviratna, 1995). In this structure the strength of each story is tuned such that simultaneous yielding will occur in each story under the 1994 UBC seismic load pattern. Thus, if this load pattern is applied in an NSP, equal story ductility demands will be predicted in every story. The dynamic analysis results demonstrate that this is not the case and that significant variations of demands over the height have to be expected. These variations are caused by higher mode effects and are not present for structures whose response is governed by the fundamental mode. To some extent the importance of higher mode effects can be captured by the LDP, which is the reason why such an analysis should be performed to supplement the NSP when higher mode effects become important. Section 2.9.2.1 identifies the conditions under which an LDP is required.

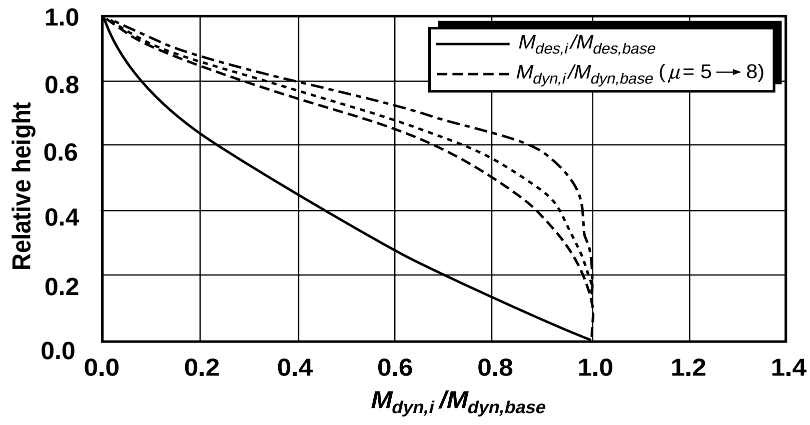
An example that demonstrates other potential problems with the NSP is that of multistory wall structures modeled by a single shear wall. In these wall structures it is assumed that the bending strength of the wall is constant over the height, and that the shear strength and stiffness are large, so that the behavior of the wall is controlled by bending. It is also assumed that no strain hardening exists once a plastic hinge has formed in the wall. The NSP will predict hinging at the base of the wall for all rational load patterns. A mechanism exists once this single plastic hinge has formed; the wall will rotate around its base, and the lateral loads can no longer be increased. Thus, the NSP will not permit propagation of plastic hinging to other stories and will predict a base shear demand that corresponds to the sum



Variation of story ductility demand over height



Amplification of base shear demand for wall structures



Story moment envelopes

Figure C3-12 Limitations of the NSP Illustrated with Story Ductility Demand, Amplification of Base Shear, and Moment Envelopes

of lateral loads needed to create the plastic hinge at the base.

Nonlinear dynamic Time-History Analysis gives very different results (Seneviratna, 1995). Higher mode effects significantly amplify the story shear forces that can be generated in the wall once a plastic hinge has formed at the base. This is illustrated in the middle diagram of Figure C3-12, which shows mean values of base shear amplification obtained by subjecting multistory wall structures to 15 ground motion records. The amplification depends on the period (number of stories) of the wall structure and on the wall bending strength (represented by μ[SDOF], the ductility ratio of the equivalent SDOF system). The diagram shows that the amplification of base shear demands may be as high as 5 for wall structures with reasonable bending strength ($\mu(\text{SDOF}) \leq 4$). This amplification implies that the base shear demand may be much higher than the base shear obtained from the lateral loads that cause flexural hinging at the base of the structure. Thus, wall shear failure may occur even though the NSP indicates flexural hinging at the base.

Nonlinear dynamic Time-History Analysis also shows that flexural hinging is not necessarily limited to the first story. It may propagate into other stories to an extent that depends on the period and flexural strength of the structure. This is illustrated in the story moment envelopes presented in the bottom diagram of Figure C3-12 for a wall structure with a period of 1.2 seconds. The moment envelope obtained from dynamic analyses is very different from that obtained from a code type load pattern (solid line).

No static analysis, whether linear or nonlinear, could have predicted this behavior. This example shows that additional measures need to be taken in some cases to allow a realistic performance assessment. Such measures need to be derived from the NDP and need to be formalized to the extent that they can be incorporated systematically in both the LSP and the NSP.

The user needs to be aware that the NSP in its present format has been based and tested on ground motions whose effects on structures can be represented reasonably by the smoothed response spectra given in Section 2.6.1 for soil classes A, B, C, and D. The prediction of the target displacement (Equation 3-11) is expected to be on the high side for soil class E. The NSP has not been tested on site-specific spectra or on near-field ground motions characterized by large displacement pulses. Moreover, the approximate modification factors contained in Equation 3-11 are calibrated for structures with a strength ratio R of about 5 or less. The modification factors may have to be increased for structures with a larger strength ratio.

C3.3.3.2 Modeling and Analysis Considerations

A. General

The general procedure for execution of the NSP is as follows.

1. An elastic structural model is developed that includes all new and old components that have significant contributions to the weight, strength, stiffness, and/or stability of the structure and whose behavior is important in satisfying the desired level of seismic performance. The structure is loaded with gravity loads in the same load combination(s) as used in the linear procedures before proceeding with the application of lateral loads.

2. The structure is subjected to a set of lateral loads, using one of the load patterns (distributions) described in the *Guidelines*. At least two analyses with different load patterns should be performed in each principal direction.

3. The intensity of the lateral load is increased until the weakest component reaches a deformation at which its stiffness changes significantly (usually the yield load or member strength). The stiffness properties of this "yielded" component in the structural model are modified to reflect post-yield behavior, and the modified structure is subjected to an increase in lateral loads (load control) or displacements (displacement control), using the same shape of the lateral load distribution or an updated shape as permitted in the *Guidelines*. Modification of component behavior may be in one of the following forms:

    a. Placing a hinge where a flexural element has reached its bending strength; this may be at the end of a beam, column, or base of a shear wall

    b. Eliminating the shear stiffness of a shear wall that has reached its shear strength in a particular story

c. Eliminating a bracing element that has buckled and whose post-buckling strength decreases at a rapid rate

d. Modifying stiffness properties if an element is capable of carrying more loads with a reduced stiffness

4. Step 3 is repeated as more and more components reach their strength. Note that although the intensity of loading is gradually increasing, the load pattern usually remains the same for all stages of the "yielded" structure, unless the user decides on the application of an adaptive load pattern (Bracci et al., 1995). At each stage, internal forces and elastic and plastic deformations of all components are calculated.

5. The forces and deformations from all previous loading stages are accumulated to obtain the total forces and deformations (elastic and plastic) of all components at all loading stages.

6. The loading process is continued until unacceptable performance is detected or a roof displacement is obtained that is larger than the maximum displacement expected in the design earthquake at the control node.

   Note: Steps 3 through 6 can be performed systematically with a nonlinear computer analysis program using an event-by-event strategy or an incremental analysis with predetermined displacement increments in which iterations are performed to balance internal forces.

7. The displacement of the control node versus first story (base) shear at various loading stages is plotted as a representative nonlinear response diagram of the structure. The changes in slope of this curve are indicative of the yielding of various components.

8. The control node displacement versus base shear curve is used to estimate the target displacement by means of Equation 3-11. Note that this step may require iteration if the yield strength and stiffnesses of the simplified bilinear relation are sensitive to the target displacement.

9. Once the target displacement is known, the accumulated forces and deformations at this displacement of the control node should be used to evaluate the performance of components and elements.

   a. For deformation-controlled actions (e.g., flexure in beams), the deformation demands are compared with the maximum permissible values given in Chapters 5 through 8.

   b. For force-controlled actions (e.g., shear in beams), the strength capacity is compared with the force demand. Capacities are given in Chapters 5 through 8.

10. If either (a) the force demand in force-controlled actions, components, or elements, or (b) the deformation demand in deformation-controlled actions, components, or elements, exceeds permissible values, then the action, component, or element is deemed to violate the performance criterion.

Asymmetry of a building in the direction of lateral loading will affect the force and deformation demands in individual components. Asymmetric elements and components in a building, such as reinforced concrete shear walls with T- or L-shaped cross section, have force and deformation capacities that may vary substantially for loading in opposite directions. Accordingly, it is necessary to perform two nonlinear procedures along each axis of the building with loads applied in the positive and negative directions, unless the building is symmetric in the direction of lateral loads or the effects of asymmetry can be evaluated with confidence through judgment or auxiliary calculations.

The recommendation to carry out the analysis to at least 150% of the target displacement is meant to encourage the engineer to investigate likely building performance under extreme load conditions that exceed the design values. The engineer should recognize that the target displacement represents a mean displacement value for the design earthquake loading, and that there is considerable scatter about the mean. Estimates of the target displacement may be unconservative for buildings with low strength compared with the elastic spectral demands. Although data are lacking at the time of this writing, it is expected that 150% of the target displacement is approximately a mean plus one standard deviation displacement value for buildings with a lateral strength in excess of 25% of the elastic spectral strength.

As noted in Step 1 of the NSP, gravity loads need to be applied as initial conditions to the nonlinear procedure, and need to be maintained throughout the analysis. This is because superposition rules applicable to linear procedures do not, in general, apply to nonlinear procedures, and because the gravity loads may importantly influence the development of nonlinear response. The gravity-load combinations are the same as in the linear procedures. As noted previously, the use of more than one gravity-load combination will greatly increase the analysis effort in the NSP. It may be possible by inspection to determine that one of the two specified combinations will not be critical.

The mathematical model should be developed to be capable of identifying nonlinear action that may occur either at the component ends or along the length of the component. For example, a beam may develop a flexural plastic hinge along the span (rather than at the ends only), especially if the spans are long or the gravity loads are relatively high. In such cases, nodes should be inserted in the span of the beam to capture possible flexural yielding between the ends of the beam. This condition is illustrated in Figure C3-13 for a simple portal frame for increasing levels of earthquake load, namely, zero (part a) to E' (part b) to E'' (part c).

B. Control Node

No commentary is provided for this section.

C. Lateral Load Patterns

The distribution of lateral inertia forces varies continuously during earthquake response. The extremes of the distribution will depend on the severity of earthquake shaking (or degree of nonlinear response), the frequency characteristics of the building and earthquake ground motion, and other aspects. The distribution of inertia forces determines relative magnitudes of shears, moments, and deformations. The loading profile that is critical for one design quantity may differ from that which is critical for another design quantity. Recognizing these aspects, design according to the NSP requires that at least two lateral load profiles be considered. With these two profiles it is intended that the range of design actions occurring during actual dynamic response will be approximately bound. Other load profiles, including adaptive load patterns, may be considered.

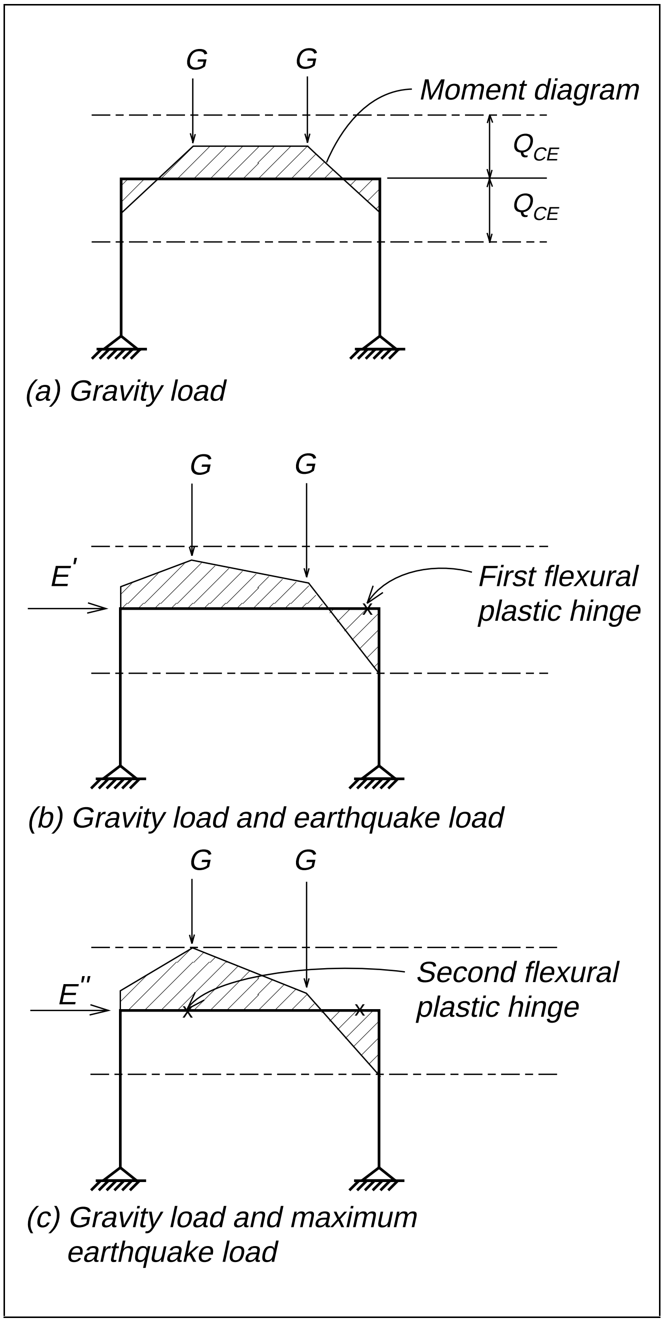


Figure C3-13 Identification of Potential Plastic Hinge Locations

Some researchers have proposed adaptive load patterns, that is, patterns that change as the structure is displaced to larger amplitude. Different suggestions have been made in this regard, including the use of story forces that are proportional to the deflected shape of the structure (Fajfar and Fischinger, 1988), the use of load patterns based on mode shapes derived from secant stiffnesses at each load step (Eberhard and Sozen, 1993), and the use of patterns in which the applied story forces are proportional to story shear resistances at each step (Bracci et al., 1995). Because these alternatives require more analysis effort and their superiority to invariant load patterns has not been demonstrated, the use of adaptive load patterns is not required in the *Guidelines*. While these adaptive patterns are not specifically identified in the *Guidelines*, one of these may be substituted for one of the specified patterns in cases where it provides a more conservative bounding load distribution than the other patterns described in the *Guidelines*.

For the time being, only very simple invariant load patterns are specified in the *Guidelines*. The “uniform” load pattern is specified because it emphasizes demands in lower stories over demands in upper stories, and magnifies the relative importance of story shear forces compared with overturning moments. The load pattern based on the coefficient C_{vx} is an option presented for simplicity and consistency with the LSP. When higher mode effects are deemed to be important, a load pattern based on modal forces combined using either the SRSS or CQC methods should also be used. Such a pattern, developed using first and second mode information, is recommended for structures whose fundamental period exceeds 1.0 second. In this manner, credit is given at least to the elastic higher-mode effects.

D. Period Determination

As a structure responds inelastically to an earthquake, the apparent fundamental period changes with response amplitude. Some researchers have proposed to estimate design responses using a fundamental period corresponding to the secant stiffness at maximum displacement. It should be recognized, however, that elastic response spectra provide only an approximation of response once a structure has entered the nonlinear range, regardless of what reference period is used. For this reason, and to simplify the analysis process, the writers have adopted a reference period corresponding to the secant stiffness at 60% of the yield strength. Determination of this period requires that the structure first be loaded laterally to large deformation levels, and that the overall load-deformation relation be examined graphically.

It is not appropriate to use empirical code period equations for T, such as those given in Section 3.3.1.2. Such equations usually provide low estimates for fundamental periods. Low estimates are appropriate for the linear procedures, because they generally result in larger spectral design forces to be applied to the mathematical model, and therefore lead to more conservative results when used with the linear procedures. On the contrary, it is more conservative to use a high estimate of fundamental period for the NSP because it will usually result in a larger target displacement.

It is recommended to evaluate the use of secant stiffness at 60% of yield strength by considering its sensitivity to component verification. The intent of the specified secant stiffness is to approximate (within the structural displacement range of zero to target displacement) the nonlinear force-displacement relationship with a bilinear relationship. The best choice may be to have approximately equal area under both curves. Note that in most cases it is more conservative to use a lower yield displacement and a lower secant stiffness.

E. Analysis of Three-Dimensional Models

No commentary is provided for this section.

F. Analysis of Two-Dimensional Models

Three-dimensional analysis models are, in principle, more appropriate than two-dimensional analysis models. However, at the time of this writing, limitations in analysis software are such that three-dimensional analysis is likely to require significantly greater analysis effort, which may not be justified for relatively symmetric buildings. Therefore, two-dimensional models may be used. The use of three-dimensional models is encouraged wherever their use is feasible.

The procedure outlined in Section 3.3.3.2F for capturing the effects of torsion is only approximate, and cannot account for the effects of inelastic torsion. Three-dimensional analysis is recommended wherever possible for buildings with either low torsional stiffness, or substantial elastic torsional response.

The rule for multidirectional excitation is adapted from Section 3.2.7 for analysis of two-dimensional models.

C3.3.3.3 Determination of Actions and Deformations

Actions and deformations in components and elements are to be calculated at a predetermined displacement of the control node. The predetermined displacement is termed the target displacement.

A. Target Displacement

The *Guidelines* present one recognized procedure for calculating the target displacement. Other procedures also can be used. This commentary presents background information on two acceptable procedures. The first procedure, here termed Method 1, is that described in the *Guidelines*. The second procedure, here termed Method 2, and commonly referred to as the Capacity Spectrum Method, is described here but not in the *Guidelines*.

Method 1. This method is presented in the *Guidelines* for the NSP. It uses data from studies of SDOF systems to determine the target displacement for a multi-degree-of-freedom (MDOF) building. Baseline data used to estimate target displacements have been derived from statistical studies on bilinear and trilinear, non-strength-degrading SDOF systems with viscous damping equal to 5% of the critical value. In order to transform the response of an MDOF building into that of an equivalent SDOF system, the nonlinear force-deformation relation determined from the NSP must be replaced by a bilinear relationship. This transformation is illustrated in Figure C3-14. Additional details on the transformation from the MDOF building to the SDOF model are provided in the supplemental information at the end of this section.

The available SDOF and MDOF studies show that the maximum displacement response of a structure responding to an earthquake ground motion is governed by many parameters. Of primary importance is the effective stiffness of the structure, as represented by K_e in the NSP. The strength is mainly important for structures with a short fundamental vibration period relative to the predominant period of the ground motion; this parameter is represented in the NSP through the strength ratio R. Pinching and strength degradation can lead to increased displacements; these effects are difficult to characterize. As such, the effects of pinching and strength degradation (that is, the shape of the hysteresis loop) are lumped together and represented by the coefficient C_2. Post-yield stiffness tends to be important only if the stiffness approaches zero or becomes negative due to either

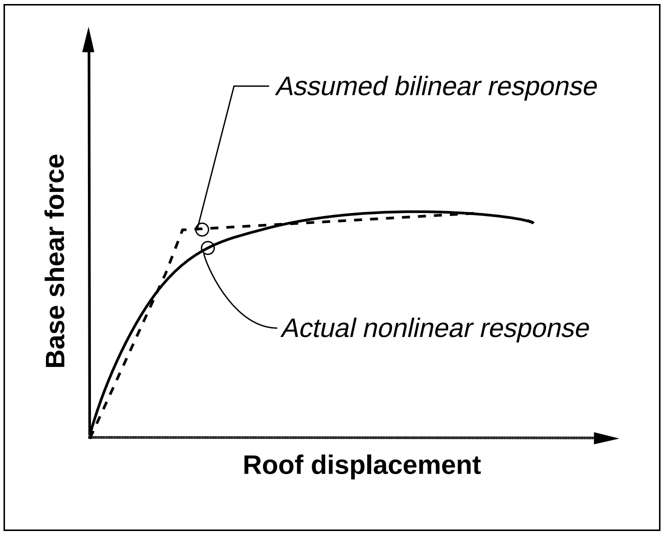


Figure C3-14 Base Shear Versus Displacement Relations

strength degradation of components or to P-Δ effects; these effects are captured approximately by coefficient C_3. The various coefficients in Equation 3-11 are discussed below.

Coefficient C_0. This coefficient accounts for the difference between the roof displacement of an MDOF building and the displacement of the equivalent SDOF system. Using only the first mode shape (ϕ_1) and elastic behavior, coefficient C_0 is equal to the first-mode participation factor at the roof (control node) level ($= \Gamma_{1,r}$):

$$C_0 = \Gamma_{1,r} = \phi_{1,r}\frac{\{\phi_1\}^T[M]\{1\}}{\{\phi_1\}^T[M]\{\phi_1\}} = \phi_{1,r}\Gamma_1 \qquad \text{(C3-3)}$$

where $[M]$ is a diagonal mass matrix, and Γ_1 is the first mode mass participation factor. Since the mass matrix is diagonal, Equation C3-3 can be rewritten as:

$$C_0 = \phi_{1,r}\frac{\Sigma_1^N m_i \phi_{i,n}}{\Sigma_1^N m_i \phi_{i,n}^2} \qquad \text{(C3-4)}$$

where m_i is the mass at level i, and $\phi_{i,n}$ is the ordinate of mode shape i at level n. If the absolute value of the roof (control node) ordinate of each mode shape is set equal to unity, the value of coefficient C_0 is equal to the first mode mass participation factor.

The actual shape vector may take on any form, particularly since it is intended to simulate the time-varying deflection profile of the building responding inelastically to the ground motion. Based on past studies, the use of a shape vector corresponding to the deflected shape at the target displacement level may be more appropriate. This shape will likely be different from the elastic first-mode shape. The use of such a deflected shape vector in the estimation of C_0 is preferred; the choice of the elastic first-mode shape vector is a simpler alternative that takes into account at least the relative mass distribution over the height of the structure; and the use of the tabulated values, which are based on a straight-line vector with equal masses at each floor level, may be very approximate, particularly if masses vary much over the height of the building.

Coefficient C_1. This coefficient accounts for the observed difference in peak displacement response amplitude for nonlinear response as compared with linear response, as observed for buildings with relatively short initial vibration periods. For use with the NSP, it is recommended to calculate the value of this coefficient using Equation 3-12. However, it is permitted to calculate this coefficient using the more approximate, and in some cases less conservative, procedure allowed for in the LSP. Limitation of the value of C_1 to the value used for the linear procedures is introduced so as not to penalize the use of the NSP. Additional discussion of this coefficient is in the commentary to Section 3.3.1.3.

Coefficient C_2. This coefficient adjusts design values based on the shape of the hysteresis characteristics of the building. See the commentary to Section 3.3.1.3 for additional discussion.

Coefficient C_3. P-Δ effects caused by gravity loads acting through the deformed configuration of a building will always result in an increase in lateral displacements. Static P-Δ effects can be captured using procedures set forth in Section 3.2.5. If P-Δ effects result in a negative post-yield stiffness in any one story, such effects may significantly increase the inter-story drift and the target displacement. The degree by which dynamic P-Δ effects increase displacements depends on (1) the ratio α of the negative post-yield stiffness to the effective elastic stiffness, (2) the fundamental period of the building, (3) the strength ratio R, (4) the hysteretic load-deformation relations for each story, (5) the frequency characteristics of the ground motion, and (6) the duration of the strong ground motion. Because of the number of parameters involved, it is difficult to capture dynamic P-Δ effects with a single modification factor. Coefficient C_3, calculated only for those buildings that exhibit negative post-yield stiffness, given by Equation 3-13, represents a substantial simplification and interpretation of much analysis data. For information, refer to Figure C3-15:the displacement amplification may become very large for bilinear small

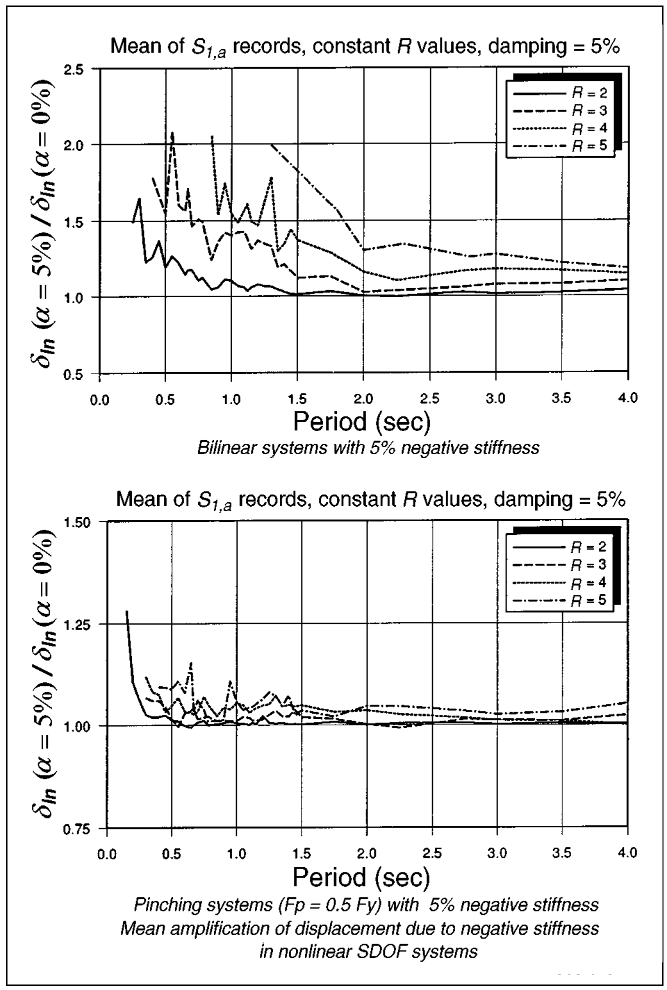


Figure C3-15 Effects of Negative Stiffness on Displacement Amplification

systems with short periods and low strength, even for values of negative stiffness (e.g., α = –0.05). The amplification is smaller for pinched hysteresis systems. Moreover, the mean results are erratic because of differences in the strong ground motions used for the analysis. The compromise offered in Equation 3-13 was to express the displacement amplification for bilinear systems by an approximate equation and to use half this value for coefficient C_3. This compromise is rationalized as follows. First, most buildings behave more like stiffness-degrading models than bilinear models. Second, in most buildings, the negative stiffness is not developed until after significant deformations have occurred. This decreases the P-Δ effects with respect to bilinear systems. However, negative stiffness in the base shear-roof displacement relation may not be representative of the negative stiffness in the critical story (likely the bottom story of the building). More work is needed in this subject area.

Method 2. Details of this procedure are not defined in the *Guidelines*, but it is considered an acceptable alternative procedure. In Method 1, the design displacement response is calculated using an initial effective stiffness. Method 2 determines maximum response based on the displacement corresponding to the intersection of the load-displacement relation (also known as the capacity curve) for the building and the spectral demand curve used to characterize the design seismic hazard. Method 2 uses initial effective stiffness and secant stiffness information to calculate the target displacement. Figure C3-16 illustrates the different stiffnesses used by the two methods, plotted in relation to the anticipated nonlinear load-displacement relation for the structure loaded to its design (target) displacement. Ideally, the two methods should produce the same design displacement. This is achieved for most cases by using different damping values for the two methods. Method 1 uses the damping effective for response near the yield level, typically 5% of the critical value. Method 2 uses a higher damping value, determined based on the shape of the hysteresis and the maximum deformation level.

This method is similar to the Capacity Spectrum Method. Further details on the Capacity Spectrum Method are in Army (1996), ATC (1982, 1996), Freeman et al. (1975), Freeman (1978), and Mahaney et al. (1993). The general procedure for using the method is similar to that for the NSP, described in the commentary on Section 3.3.3.2A. The procedure, including iterations that may be necessary, is described below.

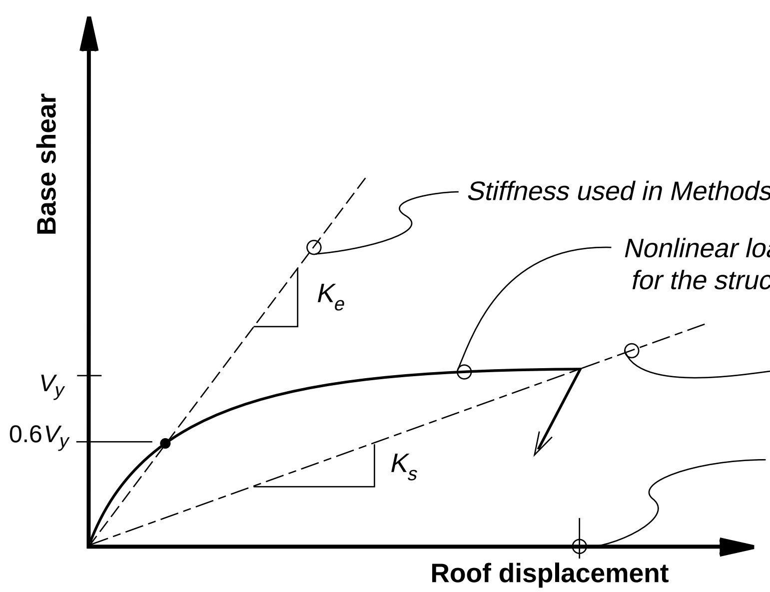


Figure C3-16 Stiffness Calculations for Estimating Building Response

Steps 1–7. These steps are identical to those described in Section C3.3.3.2A.

Step 8. The target displacement is estimated, based on either an initial assumption or information obtained from previous iterations in the procedure. Given this target displacement, an effective initial stiffness K_e is determined using procedures described in Section 3.3.3.2D. The secant stiffness K_s is defined by the slope of a line from the origin to the nonlinear load-deformation relation at the point corresponding to the target displacement. The corresponding global displacement ductility is defined as $\mu_{\delta g} = K_e / K_s$.

Step 9. The equivalent viscous damping is determined as a function of the global displacement ductility and the expected shape of the hysteresis relation for response at that ductility level using either explicit calculation (ATC, 1996) or tabulated data for different seismic framing systems (Army, 1996).

Step 10. Given the equivalent viscous damping determined as described above, a design response spectrum for that damping is constructed. As described in Section 2.6.1.5, this can be achieved by first constructing the general acceleration response spectrum for 5% damping, and then modifying it by the coefficients in Table 2-15 for different levels of damping. The acceleration response spectrum can be converted to a displacement response spectrum by multiplying the acceleration response spectrum ordinates by the factor $T^2/(4\pi^2)$. Figure C3-17 illustrates the effect of different damping levels on a typical acceleration and displacement response spectrum.

Step 11. Compare the displacement response amplitude calculated for the assumed secant stiffness and damping with the displacement amplitude assumed in Step 8. If the values differ by more than about 10%, iterate the process beginning with Step 8.

As noted in Step 10, the spectral acceleration and spectral displacement spectra are related by the factor $T^2/(4\pi^2)$. Therefore, it is possible to plot both the spectral acceleration and the spectral displacement on a single graph. Figure C3-18 plots an example for a range of equivalent viscous damping. The radial lines correspond to lines of constant period. This form of the design loading is convenient because it can be compared directly with the nonlinear load-deformation relation for the building, normalized with respect to the equivalent SDOF coordinates as described in the Supplemental Information on the NSP below. Using this format, the target displacement for the equivalent SDOF system is at the intersection of the load-deformation envelope with the response spectrum for the appropriate damping level. Note that the target displacement for the equivalent SDOF system in general is not the same as the target displacement at the roof level; to arrive at the roof level target displacement requires transformation back to the MDOF system.

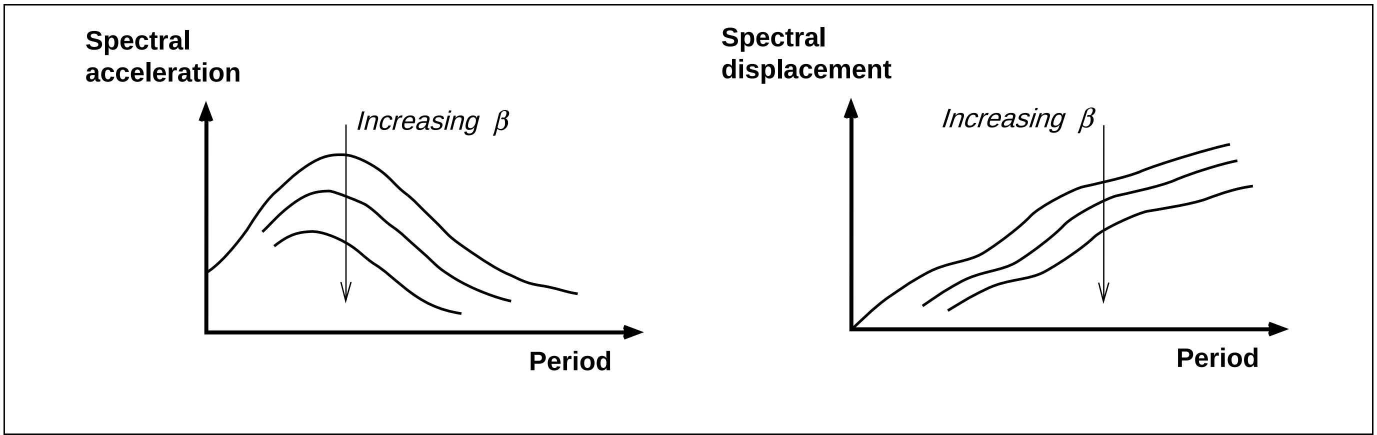


Figure C3-17 Spectral Acceleration and Displacement Curves

Supplemental Information on the NSP. The NSP is based in part on the assumption that the response of a building can be related to the response of an equivalent SDOF system. This implies that response is controlled

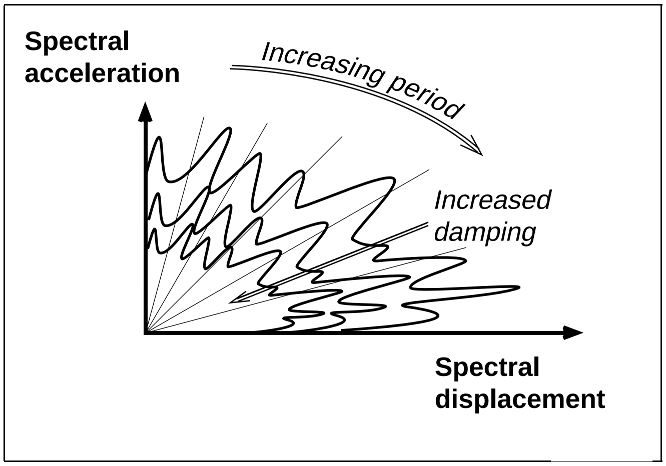


Figure C3-18 Spectral Demand Curves

by a single mode, and that the shape of this mode remains essentially constant throughout the response history. Although both assumptions are incorrect, pilot studies (Saiidi and Sozen, 1981; Fajfar and Fischinger, 1988; Qi and Moehle, 1991; Miranda, 1991; Lawson et al., 1994) have indicated that these assumptions lead to reasonable predictions of the maximum seismic response of MDOF buildings, provided response is dominated by the first mode.

The formulation of the equivalent SDOF system assumes that the deflected shape of the MDOF system can be represented by a shape vector, $\{\Phi\}$, that remains constant throughout the response history, regardless of the level of deformation. The choice of the shape vector is discussed at the end of this section. The transformation of the MDOF system to an equivalent SDOF system is derived below.

The governing differential equation of the MDOF system is:

$$[M]\{\ddot{X}\} + [C]\{\dot{X}\} + \{Q\} = -[M]\{1\}\ddot{x}_g \quad \text{(C3-5)}$$

where $[M]$ and $[C]$ are the mass and damping matrices, $\{X\}$ is the relative displacement vector, and $\ddot{x}_g$ is the ground acceleration history. Vector $\{Q\}$ denotes the story force vector. Let the assumed shape vector $\{\Phi\}$ be normalized with respect to the roof displacement, x_t; that is, $\{X\} = \{\Phi\}x_t$. Substituting this expression for $\{X\}$ in Equation C3-5 yields:

$$[M]\{\Phi\}\ddot{x}_t + [C]\{\Phi\}\dot{x}_t + \{Q\} = -[M]\{1\}\ddot{x}_g \quad \text{(C3-6)}$$

Define the SDOF reference displacement x^r as:

$$x^r = \frac{\{\Phi\}^T[M]\{\Phi\}}{\{\Phi\}^T[M]\{1\}}x_t \quad \text{(C3-7)}$$

Pre-multiplying Equation C3-6 by $\{\Phi\}^T$ and substituting for x_t using Equation C3-7 results in the governing differential equation for the response of the equivalent SDOF system:

$$M^r\ddot{x}^r + C^r\dot{x}^r + Q^r = -M^r\ddot{x}_g \quad \text{(C3-8)}$$

where:

$$M^r = \{\Phi\}^T[M]\{1\} \quad \text{(C3-9)}$$

$$Q^r = \{\Phi\}^T\{Q\} \quad \text{(C3-10)}$$

$$C^r = \{\Phi\}^T[C]\{\Phi\}\frac{\{\Phi\}^T[M]\{1\}}{\{\Phi\}^T[M]\{\Phi\}} \quad \text{(C3-11)}$$

The force-displacement relation of the equivalent SDOF system can be determined from the results of an NSP of the MDOF structure (Figure 3-1) using the shape vector established above. To identify global strength and displacement quantities, the multilinear relation is represented by a bilinear relationship that is defined by a yield strength, an average elastic stiffness ($K_e = V_y/\delta_y$), and a softening stiffness, K_s ($=\alpha K_e$). For reference, the force versus displacement relations for the MDOF system and the equivalent SDOF system are presented in Figure C3-19.

The base shear force at yield (V_y) and the corresponding roof displacement ($x_{t,y}$) from Figure C3-19 are used together with Equations C3-7 and C3-10 to compute the force-displacement relationship for the equivalent SDOF system as follows. The initial period of the equivalent SDOF system (T_{eq}) can be computed as:

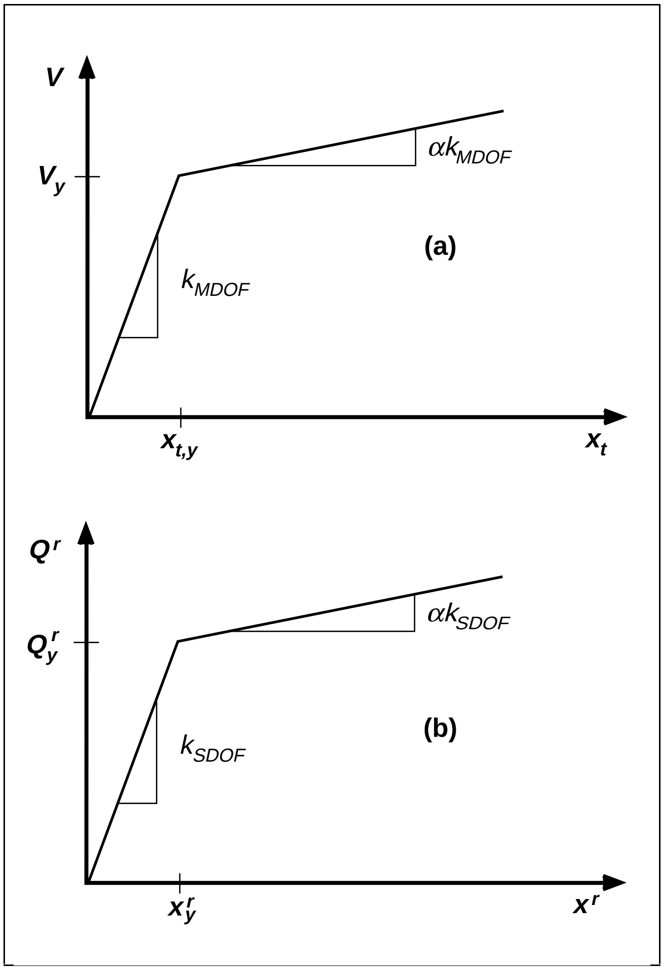


Figure C3-19 Force-Displacement Relations of MDOF Building and Equivalent SDOF System

$$T_{eq} = 2\pi\left[\frac{x_y^r M^r}{Q_y^r}\right]^{1/2} \tag{C3-12}$$

where the reference SDOF yield displacement x_y^r is calculated as:

$$x_y^r = \frac{\{\Phi\}^T[M]\{\Phi\}}{\{\Phi\}^T[M]\{\Phi\}}x_{t,y} \tag{C3-13}$$

and the reference SDOF yield force, Q_y^r, is calculated as:

$$Q_y^r = \{\Phi\}^T\{Q_y\} \tag{C3-14}$$

where $\{Q_y\}$ is the story force vector at yield, namely, $V_y = \{1\}^T\{Q_y\}$.

The strain-hardening ratio (α) of the force-displacement curve of the MDOF structure will define the strain-hardening ratio of the bilinear force-displacement curve of the equivalent SDOF system.

Using the above information, the equivalent SDOF system is now characterized. The next step in the analysis process is the calculation of the target displacement for the purpose of performance evaluation. The properties of the equivalent SDOF system, together with spectral information for inelastic SDOF systems, provide the information necessary to estimate the target displacement.

For elastic SDOF systems, the spectral displacement can be obtained directly from the design ground motion spectrum. If spectral accelerations are given, the spectral displacements S_d can be calculated as $S_a T^2/(4\pi^2)$ where T is the period of the elastic SDOF system.

Displacements of nonlinear (inelastic) SDOF systems differ from those of linearly-elastic SDOF systems, particularly in the short-period range (see Figure C3-9). In the short-period range, the ratio of inelastic to elastic displacement depends strongly on the inelastic deformation demand for the system, which is expressed in terms of the ductility ratio. The relation between the ductility ratio and the ratio of elastic to inelastic strength demands can be expressed by relationships (see Figure C3-9), which have been developed recently by several investigators (Miranda and Bertero, 1994).

Thus, to calculate a target displacement, the ductility demand for the equivalent SDOF system must be calculated. This last step requires the engineer to estimate of the ratio of elastic strength demand to yield strength of the equivalent SDOF system. Since inelastic spectra are usually obtained for unit mass systems, it is convenient to divide Equation C3-8 by M^r to obtain the differential equation for the unit mass equivalent SDOF system:

$$\ddot{x}^r + \frac{C^r}{M^r}\dot{x}^r + \frac{Q^r}{M^r} = -\ddot{x}_g \qquad \text{(C3-15)}$$

Equation C3-15 describes the response of a unit mass SDOF system with period T_{eq} and yield strength $F_{y,\,eq}$ given as

$$F_{y,\,eq} = \frac{Q_y^r}{M^r} \qquad \text{(C3-16)}$$

If the elastic response spectrum is known, the elastic strength demand of the unit mass equivalent SDOF system can be computed as:

$$F_{e,\,eq} = S_a(T_{eq}) \qquad \text{(C3-17)}$$

where the term on the right-hand side of the equation is the spectral acceleration ordinate. The strength reduction factor R can then be obtained from the relationship

$$R = \frac{F_{e,\,eq}}{F_{y,\,eq}} = \frac{S_a(T_{eq})M^r}{Q_y^r} \qquad \text{(C3-18)}$$

The ductility demand of the equivalent SDOF system can now be obtained from published $R - \mu - T$ relationships.

Note that the published data presents mean results; for essential and other important structures, the reader is encouraged to use mean plus one standard deviation displacement demands in lieu of mean displacement demands.

Since the ductility demands of the equivalent SDOF system and the MDOF structure are assumed to be equal, the target displacement of the MDOF system, $x_{t,\,t}$, is given by

$$x_{t,\,t} = \mu x_{t,\,y} \qquad \text{(C3-19)}$$

Further modifications to the target displacement may be needed to account for local soil effects, effects of strength and stiffness degradation, second-order effects, and other factors that may significantly affect displacement response.

The two key quantities needed to compute the target displacement are the period (T_{eq}) and the yield strength ($F_{y,\,eq}$) of the equivalent SDOF system. These quantities depend on the shape vector $\{\phi\}$, the story force vector $\{Q\}$, and the mass distribution over the height of the building. The need for a simplified approach makes necessary the use of readily available parameters to estimate these quantities. The first mode period (T_1) and the first mode participation factor (PF_1) are suitable for this purpose. Given the substantial variations in the shape vector, the following assumptions are made:

$$T_{eq} = T_1 \qquad \text{(C3-20)}$$

$$F_{y,\,eq} = \frac{V_y}{W}PF_1 \qquad \text{(C3-21)}$$

The accuracy of these assumptions was investigated in a sensitivity study using a triangular story force vector, equal masses at each floor, and shape vectors for wall structures—ranging from an elastic deflected shape to a straight line deflected shape (representing plastic hinging at the base and no elastic deformations). The plastic component of the roof displacement is described by the parameter p as shown in Figure C3-20. The results of the study are presented in Figure C3-21. The lower plot demonstrates the accuracy of Equation C3-21.

This study, and a companion study using shape vectors representing framed structures with story mechanisms, indicate that T_{eq} and $F_{y,\,eq}$ are insensitive to the choice of shape vector. Accordingly, the expression for the strength ratio R given by Equation 3-12 is likely adequate.

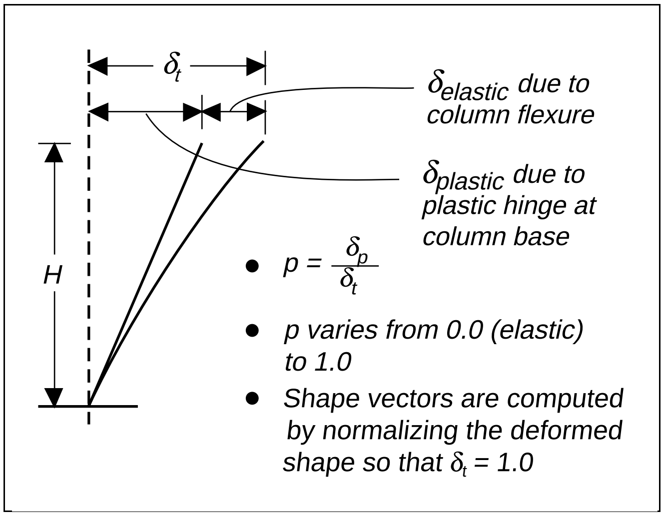


Figure C3-20 Shape Vectors used in Sensitivity Study

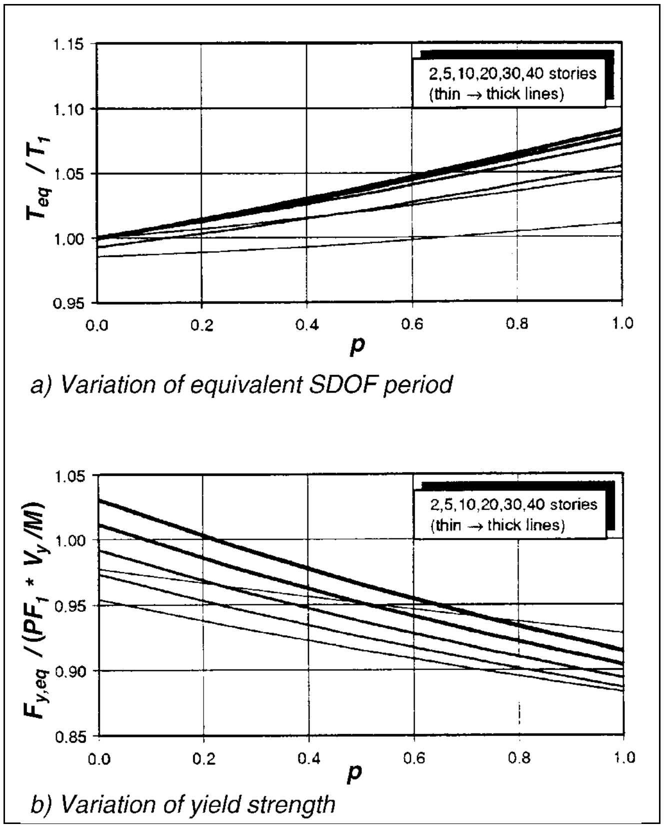


Figure C3-21 Sensitivity to the Choice of Shape Vector

B. Floor Diaphragms

Floor diaphragms shall be designed to transfer the inertia forces calculated using either of the linear procedures (Sections 3.3.1.3D or 3.3.2.3B) plus the horizontal forces resulting from offsets in, or changes in stiffness of, the vertical seismic framing elements above and below the diaphragm.

Other rational procedures may be used to calculate the inertia forces at each floor level for the purpose of diaphragm design.

C3.3.4 Nonlinear Dynamic Procedure (NDP)

C3.3.4.1 Basis of the Procedure

No commentary is provided for this section.

C3.3.4.2 Modeling and Analysis Assumptions

A. General

The modeling and analysis considerations described in Section C3.3.3.2 apply to the NDP unless superseded by provisions in Section 3.3.4.1. All masses in the building must be represented in the mathematical model and located so as to adequately capture horizontal and vertical inertial effects.

Diaphragms may be assumed to behave in the elastic range to simplify the nonlinear model. However, if the diaphragm represents the primary nonlinear element in the structural system, the mathematical model should include the nonlinear force-deformation characteristics of the diaphragm (Kunnath et al., 1994).

B. Ground Motion Characterization

Ground motion time-histories are required for the NDP. Such histories (or pairs thereof) shall be developed according to the requirements of Section 2.6.1.

C. Time-History Method

See Section C3.3.2.2D for pertinent information.

C3.3.4.3 Determination of Actions and Deformations

A. Modification of Demands

The element and component deformations and actions used for evaluation shall be established using the results of the NDP.

C3.4 Acceptance Criteria

C3.4.1 General Requirements

No commentary is provided for this section.

C3.4.2 Linear Procedures

These acceptance criteria apply for both the LSP and the LDP. (See Section C3.4.2.2A for supplemental information on linear procedures acceptance criteria and Equation 3-18.)

C3.4.2.1 Design Actions

This section defines the actions (forces and moments), including gravity and earthquake effects, for which the evaluation is carried out.

A. Deformation-Controlled Actions

Equation 3-14 defines the deformation-controlled actions for design. This equation states the design actions in force terms, although the intent is to provide an indirect (albeit very approximate) measure of the deformations that the structural component or element experiences for the combination of design gravity loading plus design earthquake loading. Because of possible anticipated nonlinear response of the structure, the design actions as represented by this equation may exceed the actual strength of the component or element to resist these actions. The acceptance criteria of Section 3.4.2.2A take this overload into account through use of a factor, m, which is an indirect measure of the nonlinear deformation capacity of the component or element.

B. Force-Controlled Actions

The basic approach for calculating force-controlled actions for design differs from that used for deformation-controlled actions. The reason is that, whereas nonlinear deformations may be associated with deformation-controlled actions, nonlinear deformations associated with force-controlled actions are not permitted. Therefore, force demands for force-controlled actions must not exceed the force capacity (strength).

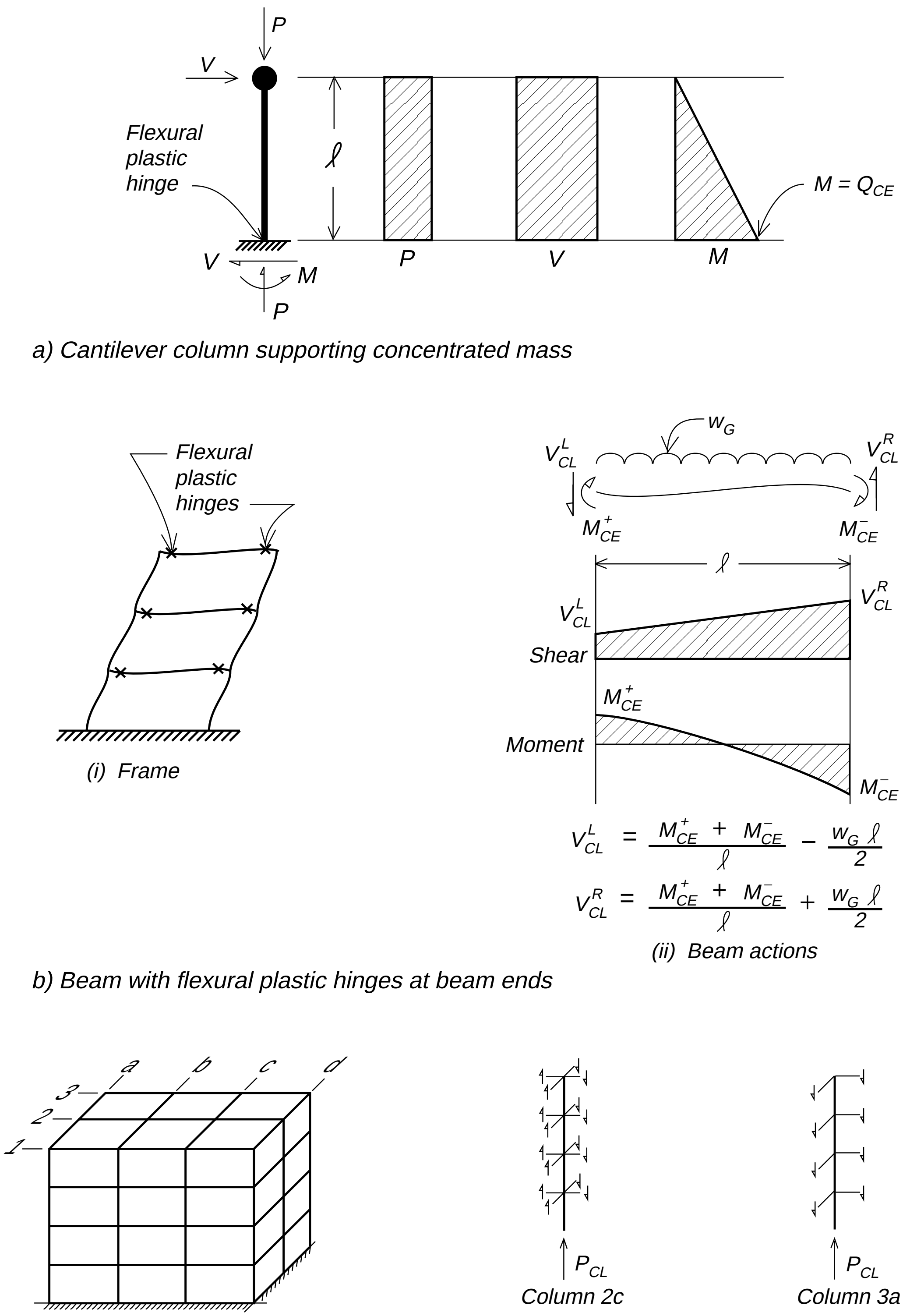
Ideally, an inelastic mechanism for the structure will be identified, and the force-controlled actions Q_{UF} for design will be determined by limit analysis using that mechanism. This approach will always produce a conservative estimate of the design actions, even if an incorrect mechanism is selected. Where it is not possible to use limit (or plastic) analysis, or in cases where design forces do not produce significant nonlinear response in the building, it is acceptable to determine the force-controlled actions for design using Equations 3-15 and 3-16. Additional discussion of both approaches is provided below.

Limit analysis to determine force-controlled design actions is relatively straightforward for some components and some structures. The concept is illustrated in a series of structural idealizations in Figure C3-22. Each of these cases is discussed briefly in the paragraphs below.

Figure C3-22(a) illustrates a structure consisting of a single cantilever column with a mass at the top. The deformation-controlled action is flexure at the column base. Force-controlled actions include axial load and shear force. Assuming a nonlinear mechanism involving flexure at the base of the column, and using the expected moment strength Q_{CE} at that location, the design shear force is calculated from equilibrium to be equal to Q_{CE}/l, where l is the column length. Because earthquake loading produces no axial force in this column, the design axial force is equal to the gravity level value.

Figure C3-22(b) illustrates a multistory frame. Considering a typical beam, the deformation-controlled actions are flexural moment at the beam ends, and the force-controlled action of interest is the beam shear. Assuming a nonlinear mechanism involving flexure at the beam ends, and using the expected moment strengths Q_{CE} at those locations, the design shear force at various locations along the beam can be calculated from equilibrium of a free-body diagram loaded by the expected moment strengths and gravity loads. This same approach can be used to determine the design shear force in columns of frames.

Note that beam flexural moment along the length of the beam may also be assumed to be a force-controlled action because flexural yielding is not desired away from the beam ends. The beam moment diagram, determined from equilibrium of the free-body diagram, identifies the appropriate moments to be checked against the beam moment strength along the beam span.



a) Cantilever column supporting concentrated mass

b) Beam with flexural plastic hinges at beam ends

c) Column axial load determined as the sum of the beam shears. Note that in the limit, the beam shear is equal to the shear corresponding to beam flexural hinging (see part b).

Figure C3-22 Checking for Force-Controlled Actions

Figure C3-22(c) illustrates a multistory frame. Considering interior and exterior columns, the deformation-controlled actions are flexural moment at the column ends, and the force-controlled actions of interest are column shear and axial load. Assume for this example that we are interested in identifying the column axial load for design. A mechanism suitable for obtaining design axial loads is shown. A free-body diagram of each column is made by making a cut at the intersection with each beam framing into the column, and replacing the beam by the internal forces (moment and shear) that would be acting in the beam at that location (these actions were discussed in the previous example). Note that beams may be framing into the column from two orthogonal framing directions, and that the actions from each beam should be considered. This aspect is especially important for corner columns of frames.

Limit analysis can be used for a broad range of other cases, and specialized mechanisms can be identified that may result in reductions in the design actions that need to be considered.

Equations 3-15 and 3-16 are recommended only for those cases where it is not feasible to determine force-controlled actions for design using limit analysis, or for cases where significant levels of nonlinear action are not anticipated for the design loading.

Equations 3-15 and 3-16 are conservative and can be used to calculate all force-controlled actions. Equation 3-15 can be used to calculate actions that result from forces delivered by yielding components. For instance, it could be used to calculate the axial forces in Columns 2c and 3a (Figure C3-22) wherein the seismic axial forces are delivered by beams yielding in flexure. However, if some of the beams framing into Column 2c do not yield, Equation 3-15 cannot be used and either limit analysis or Equation 3-16 must be used to calculate the design axial force. Other examples of this condition could include pier and spandrel components in pierced shear walls, secondary components and elements, and joints and columns in slab-column framing systems.

The writers recognize that Equation 3-15 is a relatively crude estimator of actual expected forces, and therefore the equation has been defined to produce conservative results in most cases. The rationale used to develop Equation 3-15 follows.

The coefficient J in Equation 3-15 was the subject of much debate in the development of the *Guidelines*, and the final result may not be appropriate in certain cases. According to Equation 3-17, for zones of high seismicity, the value of J may equal 2.0. The result in Equation 3-17 is that the force-controlled action for design is equal to the gravity load action plus half the seismic action calculated by the linear procedures, implying that the structure has sufficient strength to resist only about half the design lateral forces. It is anticipated that most structures in regions of low seismicity will be able to resist the design seismic forces without significant yielding. Therefore, Equation 3-17 has been written so that J will reduce to unity as the spectral acceleration reduces.

Coefficient C_1 in Equations 3-15 and 3-16 is the same coefficient introduced in Equation 3-6. It was introduced in Equation 3-6 to amplify the design base shear to achieve a better estimate of the maximum displacement for short-period buildings responding in the nonlinear range. Of course, for nonlinear response, the base shear will decrease rather than increase. Thus, in most cases it is reasonable to divide this component back out of the force estimate when seeking forces using Equations 3-15 and 3-16. Coefficients C_2 and C_3 in Equations 3-15 and 3-16 were introduced in Equation 3-6 to increase the pseudo lateral load to capture the effects on maximum displacement response due to pinching and strength degradation, and second-order effects, respectively. None of these three effects will increase the base shear force. As such, these coefficients are divided back out of the seismic force estimate of Equations 3-15 and 3-16.

C3.4.2.2 Acceptance Criteria for Linear Procedures

A. Deformation-Controlled Actions

In the linear procedures of Sections 3.3.1 and 3.3.2, a linearly-elastic model of the structure is loaded by lateral forces that will displace the model to displacements expected in the building as it responds to the design earthquake. If the building responds nonlinearly, as is often the case, the lateral forces and corresponding internal forces will exceed yielding values. The degree to which the calculated internal forces exceed the component strengths is used as a measure of the extent of nonlinear deformations that develop in the component. The acceptance criteria for deformation-controlled actions, as expressed by

Equation 3-18, are based on this concept. In Equation 3-18, the design actions Q_{UD} may exceed the actual strength of the component, Q_{CE}. The modifier *m* in Equation 3-18 provides a measure of the ductility capacity of the component associated with the expected inelastic deformation mode. Figure C3-23 illustrates the *m* factor for a moment-rotation (M-ϕ) deformation-controlled action on a component or element. (Note: The *m* factor is also applicable for axial and shear deformations.) M_e (or in the notation of Equation 3-14, Q_{UD}) is the design moment (action) due to gravity loads and earthquake loads that the component or element would experience if the component or element were to remain elastic. $M_{CE} = Q_{CE}$ is the expected strength of the component or element at the expected deformation of the component or element. Thus, $m = Q_{UD}/Q_{CE}$ or $mQ_{CE} = Q_{UD}$. In Chapters 4 through 8, *m* factors are given for determining the acceptability of various soil foundation, steel, concrete, masonry, and wood components or elements. Chapter 8 also includes *m* factors for wood connections. The derivation of Equation 3-18 is provided below.

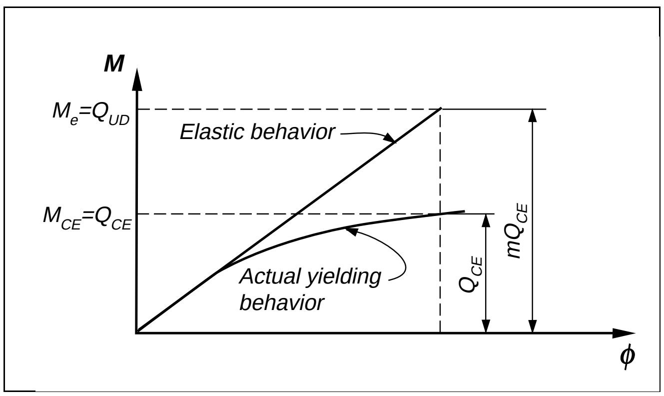


Figure C3-23 Basis for m Factor (using M as Representative of a Deformation-Controlled Action)

The expected strength of the component or element, Q_{CE}, should be calculated as the largest resistance obtained for deformations up to and including the maximum deformations to be experienced by the component for the design earthquake loading. Its calculation should take into consideration actual material properties, including strain hardening, and actual cross sections, including composite action with interconnected materials where appropriate. Procedures for calculation of Q_{CE} are specified in Chapters 5 through 8.

Note that all secondary components and elements, which are required to be excluded from the mathematical model when using the linear procedures, must be checked to ensure that they have adequate deformation capacity. This can either be done directly for each component or element where drift capacities are known, or alternately, a secondary mathematical model can be constructed that includes the secondary components. This model is subjected to the design displacements obtained for the linear procedure. All deformation-controlled actions are then checked according to Equation 3-18.

Supplemental Information on Linear Procedure Acceptance Criteria and Equation 3-18. Equation 3-18 sets the acceptance criterion for deformation-controlled actions. This equation is a displacement-based check that is expressed in force units for ease of implementation. In Equation 3-14, the gravity force actions (Q_G) calculated using Equations 3-2 and 3-3 are combined with the seismic force actions (Q_E), calculated using either Equation 3-6 for the LSP or Section 3.3.3 for the LDP. The resulting action is then compared with the expected capacity of the component that is increased by a component demand modifier, *m*.

Figures C3-24 and C3-25 illustrate the intent of Equation 3-18. The subject frame in these figures is a one-bay portal frame. It is assumed that gravity loads are applied to the beam only, and seismic inertial loads are only developed at the level of the beam. The internal actions in the beam and columns, resulting from the application of the gravity and seismic loads, are indicated in Figures C3-24 and C3-25, respectively. The following formulation assumes a statistical relation between inelastic and elastic displacements.

First, the beam is considered. Assumed loads and actions, and key response histories are indicated in Figure C3-24. It is assumed that flexure in the beam is designated as deformation-controlled. Shear and axial load effects are to be ignored. The history of the beam (and the frame) begins at point “a”. Under gravity loads (loading to point “b”), the lateral displacement of the beam is zero, while the moment at the beam end increases from zero to M_G. The beam flexural

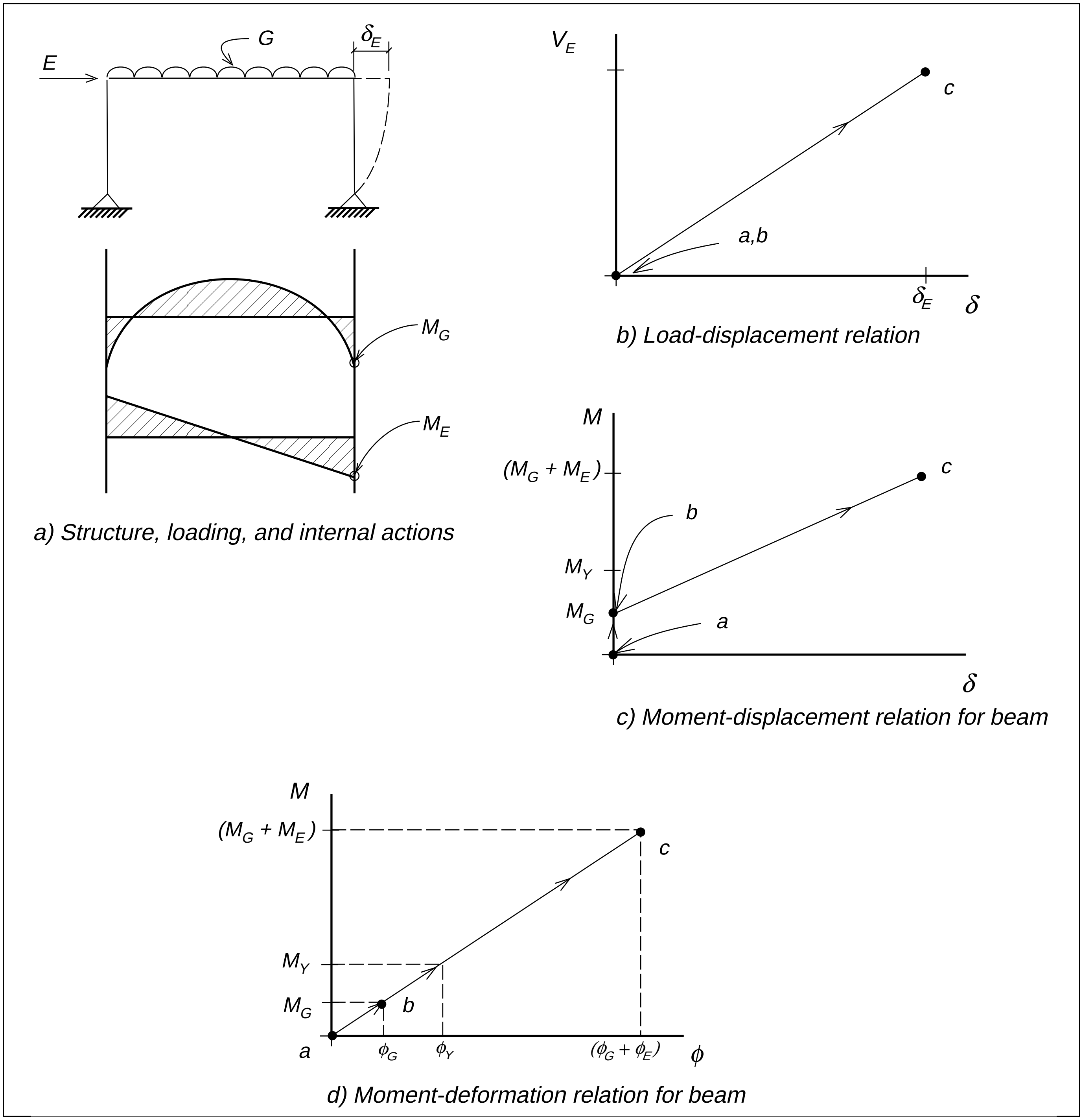


Figure C3-24 Frame Evaluation - Beam Information

deformation, expressed in the figure as ϕ, increases from zero to ϕ_G. Under lateral earthquake loading (loading to point "c"), the moment at the beam end increases from M_G to $(M_G + M_E)$. The beam deformation increases from ϕ_G to $(\phi_G + \phi_E)$. Assuming that the beam deformation increases linearly from zero to $\phi_G + \phi_E$—which will likely not quite happen because of the difference in the curvature distributions for gravity and seismic loading—it can be written that:

$$\mu_\phi = \frac{\phi_G + \phi_E}{\phi_Y} = \frac{M_G + M_E}{M_Y} \qquad \text{(C3-22)}$$

where μ_ϕ is component ductility expressed in terms of ϕ, and ϕ_Y and M_Y refer to component yield deformation and force, respectively. Reorganizing the terms in Equation C3-22 results in:

$$M_G + M_E = \mu_\phi M_Y \qquad \text{(C3-23)}$$

Equation C3-23 is essentially Equation 3-18 with μ_ϕ replacing the component demand modifier, *m*, and Q_{UD} replacing the sum of M_G and M_E.

Second, the columns in the sample frame are considered. Assumed loads and actions, and key response histories are indicated in Figure C3-25. It is assumed that flexure in the column is designated as a deformation-controlled action. The history of the column begins at point "a". Under gravity loads (loading to point "b"), the lateral displacement is zero, while the moment at the column end increases from zero to M_G, and the axial load increases from zero to P_G. The column deformation, expressed as ϕ, increases from zero to ϕ_G. Under lateral earthquake loading (loading to point "c"), the moment at the beam end increases from M_G to $(M_G + M_E)$. The column deformation increases from ϕ_G to $(\phi_G + \phi_E)$. It is clear that column deformation and column moment follow a similar path to those of the beam described above. As such, Equation 3-18 applies to the column moment. However, this equation may not apply to the axial load—a quantity that is needed to calculate M_Y. Rather, axial load may follow a very different path, and a different procedure is required to calculate it.

Equations 3-15 and 3-16 are an attempt to provide a simple and conservative estimate of the forces that occur in a component under gravity and earthquake loading. These equations should be used unless the engineer carries out limit analysis of the frame to calculate the axial load that exists when the frame is displaced to cause yielding of all actions contributing to the axial force in the members—the preferred solution method. Refer to Figure C3-26, which considers both an interior column and an exterior column. The history of this frame begins at point "a". Under gravity loads (loading to point "b"), the lateral displacement is zero, while the axial force increases to P_G. Under the application of the equivalent base shear (equal to V for the LSP), the lateral displacement increases to δ_e (loading to point "c"). The axial load in the interior column remains constant, while the axial load in the exterior column computed from the linear elastic model increases to $(P_G + P_E)$. However, because of yielding in the building frame, the maximum base shear is unlikely to reach the equivalent base shear. As the frame begins to yield, it is also likely that the axial load in the exterior column will increase at a decreasing rate. Without carrying out a detailed analysis, it is virtually impossible to pinpoint what will be the axial load in the exterior column. Meanwhile, the interior column is probably carrying the gravity axial load (P_G).

Considering the interior column, it is apparent that Equation 3-18 does not apply to the axial load on the column. In the absence of limit analysis data, Equations 3-15 and 3-16 should be used to establish the axial force coexisting with the column moment for checking the acceptability of the column.

B. Force-Controlled Actions

The lower-bound strength of the component or element, Q_{CL}, should be calculated as a mean minus one standard deviation level of resistance, taking into consideration degradation that might occur over the range of deformation cycles to which the component or element may be subjected. Procedures for calculation of Q_{CL} are specified in Chapters 5 through 8.

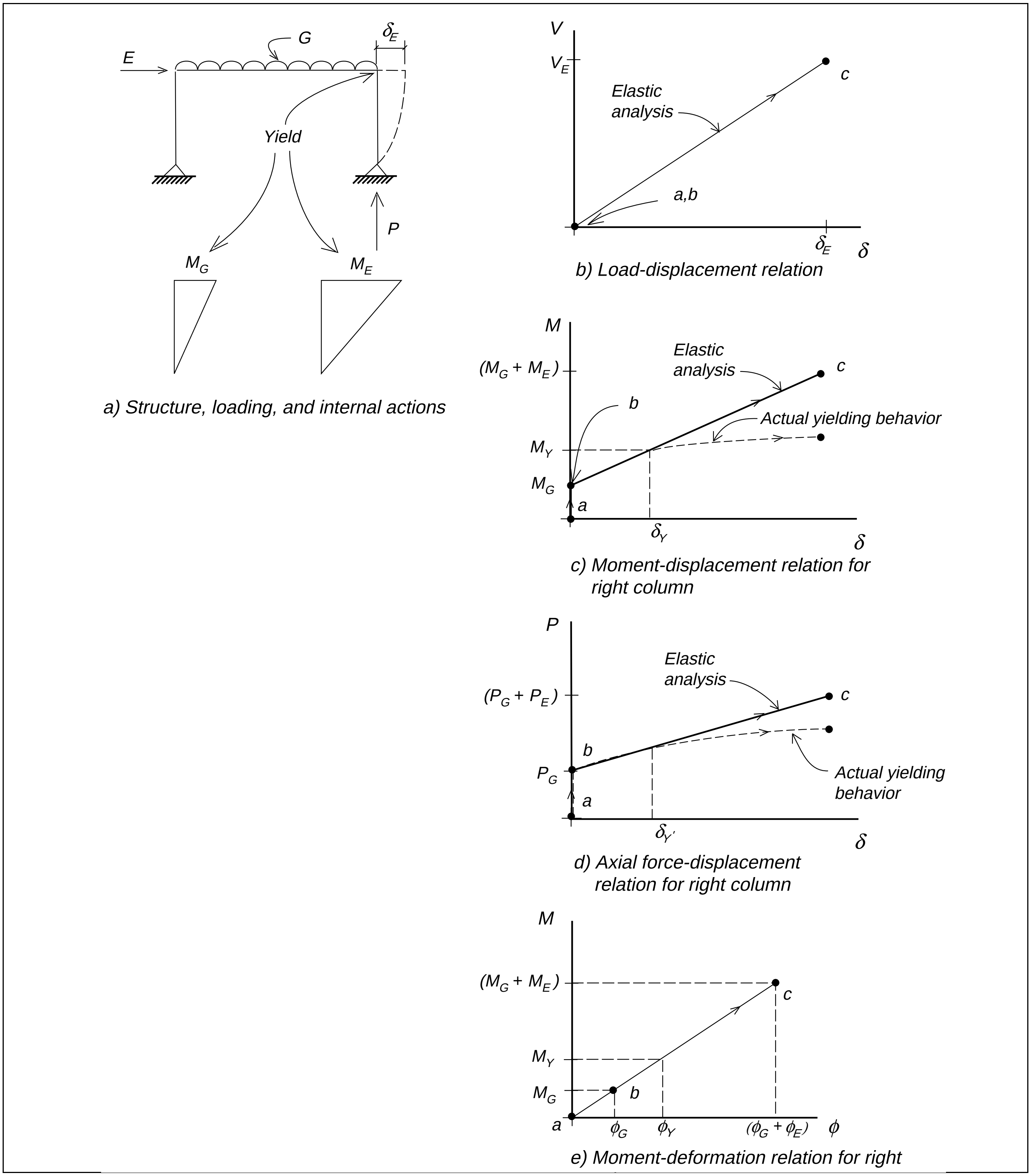


Figure C3-25 Frame Evaluation - Column Information

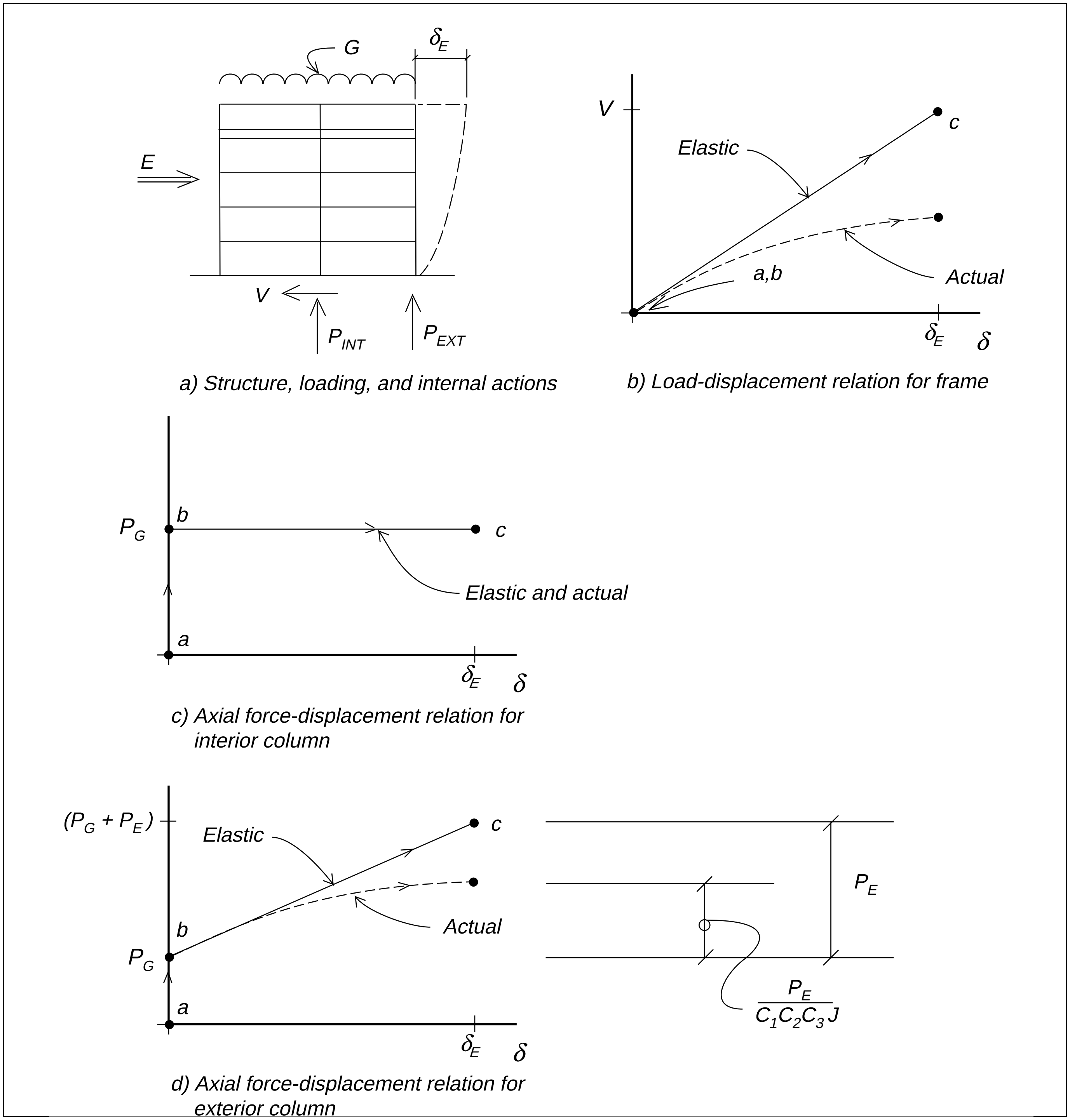


Figure C3-26 Evaluation of a Multibay Frame

Note that all secondary components and elements, which are required to be excluded from the mathematical model when using the linear procedures, must be checked to ensure that they have adequate deformation capacity. This can be done either directly for each component or element where drift capacities are known, or alternatively, a secondary mathematical model can be constructed that includes the secondary components. This model is subjected to the design displacements obtained for the linear procedure. All force-controlled actions are then checked according to Equation 3-19.

C. Verification of Design Assumptions

A primary goal of this section is to ensure that the engineer checks design actions and associated strengths at all locations within a component, rather than just at end points or at nodes used to define the component in a computer model of the building. For example, it is inadequate to check for flexural strength only at the ends of a beam; it is also necessary to check flexural design actions against flexural strengths at other locations on the beam.

For beams evaluated or designed using the linear procedures, it is required that inelastic flexural action be restricted to component ends. This is because the linear procedures can lead to nonconservative results, and may completely misrepresent actual behavior, when flexural yielding occurs along the span length. To check for this case, construct a free-body diagram of the beam loaded at its ends with the expected moment strengths Q_{CE} and along its length with the design gravity loads (Figure C3-13). The moment diagram along the length of the beam can then be constructed from equilibrium principles. The moments along the length of the beam are then compared with the strengths at all locations. For this purpose, the strength may be calculated as Q_{CE} (that is, assuming expected strength rather than lower-bound strength). Where this comparison indicates that flexural strength may be reached at locations more than one beam depth from the beam ends, either the beam should be rehabilitated to prevent inelastic action along the length, or the design should be based on one of the nonlinear procedures (Section 3.3.3 or 3.3.4).

C3.4.3 Nonlinear Procedures

These acceptance criteria apply for both the NSP and the NDP.

C3.4.3.1 Design Actions and Deformations

The NSP and the NDP both provide direct information on force and deformation demands that are associated with the specified design loading. Therefore, it is not necessary to define design forces and deformations for deformation-controlled actions and force-controlled actions using the procedures described for the linear procedures.

C3.4.3.2 Acceptance Criteria for Nonlinear Procedures

Performance evaluation consists of a capacity/demand evaluation of relevant parameters (actions and deformations). Demands are determined directly from the nonlinear procedure. Procedures for determining force and deformation capacities are specified in Chapters 5 through 8.

It must be recognized that capacity may take on a different meaning for different Performance Levels and different deformation levels. In general, strength capacities are calculated according to procedures in Chapters 4 through 8, taking into consideration the deformation level experienced by the component. Different deformation levels are permitted depending on the Performance Level.

Deformation capacities in Chapters 5 through 8 are specified in tabular form in terms of quantities that are commonly available from nonlinear analysis computer programs. At the component level, these deformations are specified in absolute terms, as plastic hinge rotation capacity, shear distortion capacity, and inter-story drift capacity. Ductility ratios are not generally used, since it may be more difficult to interpret the output data from most computer programs in these terms.

It must be recognized that at the time of this writing, neither deformation demands nor deformation capacities can be predicted accurately using the nonlinear procedures, although these procedures are generally believed to be far superior to the linear procedures in this regard. The inability to make accurate predictions may not be a major drawback, because accurate predictions usually are not critical, particularly for components that deteriorate in a gradual manner. Collapse and life-safety hazards are caused primarily by brittle failure modes in components and connections that are important parts of the gravity and lateral load paths. Thus the emphasis (with a focus on the Life Safety Performance Level) needs to be on verification of the following:

1. A complete and adequate load path exists.
2. The load path remains sound at the deformations associated with the target displacement level.
3. Critical connections remain capable of transferring loads between the components that form part of the load path.
4. Individual components that may fail in a brittle mode and that are important parts of the load path are not overlooked (where multiple failure modes are possible, ensuring that each is identified).
5. Localized failures (should they occur) do not violate the goals of the Performance Level; in particular, it must be verified that the loads tributary to the failed components can be transferred safely to other components and that the failed component itself does not pose an unacceptable hazard.
6. Finally, there should be verification of reasonable deformation control. Story drift quantities indicated in Table 2-4 may be used for reference.

C3.5 Definitions

No commentary is provided for this section.

C3.6 Symbols

No commentary is provided for this section.

C3.7 References

Army, 1996, *Seismic Dynamic Analysis for Buildings*, Final Draft, Departments of the Army (TM5-809-10-1), Navy (NAVFAC P355.1), and Air Force (AFM 88-3, Chapter 13, Section A), Washington, D.C.

ASCE, 1986, *ASCE Standard for the Seismic Analysis of Safety-Related Nuclear Structures and Commentary*, American Society of Civil Engineers, New York, New York.

ATC, 1982, *An Investigation of the Correlation Between Earthquake Ground Motion and Building Performance*, Report No. ATC-10, Applied Technology Council, Redwood City, California.

ATC, 1984, *Tentative Provisions for the Development of Seismic Regulations for Buildings*, second printing, including proposed amendments, Report No. ATC-3-06, Applied Technology Council, Redwood City, California.

ATC, 1996, *Recommended Methodology for Seismic Evaluation and Retrofit of Existing Concrete Buildings*, Draft, Report No. ATC-40, Applied Technology Council, Redwood City, California.

Bertero, V. V., 1996, “State-of-the-Art Report on Design Criteria,” *Proceedings of the Eleventh World Conference on Earthquake Engineering*, Acapulco, Mexico.

Bertero, V. V., Anderson, J. C., Krawinkler, H., and Miranda, E., 1991, *Design Guidelines for Ductility and Drift Limits*, Report No. EERC 91/15, Earthquake Engineering Research Center, University of California, Berkeley, California.

Bonacci, J. F., 1989, *Experiments to Study Seismic Drift of RC Structures*, Ph.D. diss., Graduate College, University of Illinois, Urbana, Illinois.

Bracci, J. M., Kunnath, S. K., and Reinhorn, A. M., 1995, “Simplified Seismic Performance and Retrofit Evaluation,” submitted to the *Journal of the Structural Division*, American Society of Civil Engineers, New York, New York.

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.

Clough, R., and Penzien, J., 1993, *Dynamics of Structures*, McGraw-Hill, New York, New York.

Constantinou, M. C., Soong, T. T., and Dargush, G. F., 1996, *Passive Energy Dissipation Systems for Structural Design and Retrofit*, National Center for Earthquake Engineering Research, Buffalo, New York.

Eberhard, M. O., and Sozen, M. A., 1993, "Behavior-Based Method to Determine Design Shear in Earthquake-Resistant Walls," *Journal of the Structural Division*, American Society of Civil Engineers, New York, New York, Vol. 119, No. 2, pp. 619–640.

EERC, 1995, *Seismological and Engineering Aspects of the 1995 Hyogoken-Nanbu (Kobe) Earthquake*, Report No. EERC 91/15, Earthquake Engineering Research Center, University of California, Berkeley, California.

EERI, 1996, Northridge Earthquake Reconnaissance Report, *Earthquake Spectra*, Earthquake Engineering Research Institute, Oakland, California, Vol. 2, Supp. C.

Fajfar, P., and Fischinger, M., 1988, "N2 - A Method for Non-Linear Seismic Analysis of Regular Structures," *Proceedings of the Ninth World Conference on Earthquake Engineering*, Tokyo-Kyoto, Japan.

Fajfar, P., and Krawinkler, H., (Editors), 1992, *Nonlinear Seismic Analysis and Design of Reinforced Concrete Buildings*, Elsevier Applied Science, London and New York.

Freeman, S. A., Nicoletti, J. P., and Tyrell, J. V., 1975, "Evaluation of Existing Buildings for Seismic Risk—A Case Study of Puget Sound Naval Shipyard, Bremerton, Washington," *Proceedings of the First U.S. National Conference on Earthquake Engineering*, Earthquake Engineering Research Institute, Oakland, California.

Freeman, 1978, "Prediction of Response of Concrete Buildings to Severe Earthquake Motion," *Douglas McHenry International Symposium on Concrete and Concrete Structures*, SP-55, American Concrete Institute, Detroit, Michigan, pp. 589–605.

Gulkan, P., and Sozen, M. A., 1974, "Inelastic Response of Reinforced Concrete Structures to Earthquake Motions," *Journal of the American Concrete Institute*, Detroit, Michigan, pp. 604–610.

Healey, T. J., and Sozen, M.A., 1978, "Experimental Study of the Dynamic Response of a Ten-Story Reinforced Concrete Frame with a Tall First Story," *Civil Engineering Studies, Structural Research Series*, University of Illinois, Urbana, Illinois, No. 450.

Krawinkler, H., 1994, "New Trends in Seismic Design Methodology," *Proceedings of the Tenth European Conference in Earthquake Engineering*, Vienna, Austria.

Krawinkler, H., and Nassar, A. A., 1992, "Seismic Design Based on Ductility and Cumulative Damage Demands and Capacities," *Nonlinear Seismic Analysis and Design of Reinforced Concrete Buildings*, edited by Fajfar, P., and Krawinkler, H., Elsevier Applied Science, London and New York.

Krawinkler, H., and Rahnama, M., 1992, "Effects of Soft Soils on Design Spectra," *Proceedings of the Tenth World Conference on Earthquake Engineering*, Madrid, Spain, Vol. 10.

Kunnath, S., Mehrain, M., and Gates, W., 1994, "Seismic Damage-Control Design of Gypsum-Roof Diaphragms," *Journal of the Structural Division*, American Society of Civil Engineers, New York, New York, Vol. 120, No. 1.

Lawson, R. S., Vance, V., and Krawinkler, H., 1994, "Nonlinear Static Push-Over Analysis—Why, When, and How?" *Proceedings of the Fifth U.S. Conference in Earthquake Engineering*, Earthquake Engineering Research Institute, Oakland, California, Vol. 1.

Mahaney et al., 1993, "The Capacity Spectrum Method for Evaluating Structural Response during the Loma Prieta Earthquake," *Proceedings of the National Earthquake Conference*, Memphis, Tennessee.

Mehrain, M., and Graf, W., 1990, "Dynamic Analysis of Tilt-Up Buildings," *Proceedings of the Fourth U.S. National Conference on Earthquake Engineering*, Palm Springs, California.

Miranda, E., 1991, *Seismic Evaluation and Upgrading of Existing Buildings*, Ph.D. diss., Department of Civil Engineering, University of California, Berkeley, California.

Miranda, E., 1993, "Evaluation of Site-Dependent Inelastic Seismic Design Spectra," *Journal of the Structural Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 119, No. 5, pp. 1319–1338.

Miranda, E., and Bertero, V. V., 1994, "Evaluation of Strength Reduction Factors for Earthquake-Resistant Design," *Earthquake Spectra*, Earthquake Engineering Research Institute, Oakland, California, Vol. 10, No. 2.

Moehle, J. P., 1992, "Displacement-Based Design of RC Structures Subjected to Earthquakes," *Earthquake Spectra*, Earthquake Engineering Research Institute, Oakland, California, Vol. 8, No. 3, pp. 403–428.

Nassar, A. A., and Krawinkler, H., 1991, *Seismic Demands for SDOF and MDOF Systems*, Report No. 95, John A. Blume Earthquake Engineering Center, Department of Civil Engineering, Stanford University, Stanford, California.

Nassar, A. A., Krawinkler, H., and Osteraas, J. D., 1992, "Seismic Design Based on Strength and Ductility Demands," *Proceedings of the Tenth World Conference on Earthquake Engineering*, Madrid, Spain, Vol. 10, pp. 5861–5866.

Osteraas, J. D., and Krawinkler, H., 1990, *Strength and Ductility Considerations in Seismic Design*, Report No. 90, John A. Blume Earthquake Engineering Research Center, Department of Civil Engineering, Stanford University, Stanford, California.

Popov, E., Yang, T., and Grigorian, C., 1993, "New Directions in Structural Seismic Design," *Earthquake Spectra*, Earthquake Engineering Research Institute, Oakland, California, Vol. 9, No. 4, pp. 845–875.

Qi, X., and Moehle, J. P., 1991, *Displacement Design Approach for Reinforced Concrete Structures Subjected to Earthquakes*, Report No. EERC 91/02, Earthquake Engineering Research Center, University of California, Berkeley, California.

Rahnama, M., and Krawinkler, H., 1993, *Effects of Soft Soils and Hysteresis Models on Seismic Design Spectra*, Report No. 107, John A. Blume Earthquake Engineering Center, Department of Civil Engineering, Stanford University, Stanford, California.

Saiidi, M., and Sozen, M. A., 1981, "Simple Nonlinear Seismic Analysis of R/C Structures," *Journal of the Structural Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 107, No. ST5, pp. 937–951.

Seneviratna, G.D.P.K., and Krawinkler, H., 1994, "Strength and Displacement Demands for Seismic Design of Structural Walls," *Proceedings of the Fifth U.S. National Conference on Earthquake Engineering*, Chicago, Illinois.

Seneviratna, G.D.P.K., 1995, *Evaluation of inelastic MDOF effects for seismic design*, Ph.D. diss., Department of Civil Engineering, Stanford University, Stanford, California.

Uang, Chia-M., 1991, "Establishing R (or R_w) and C_d Factors for Building Seismic Provisions," *Journal of the Structural Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 117, No. 1, pp. 19–28.

Uang, Chia-M., and Maarouf, A., 1992, "Evaluation of the Displacement Amplification Factor for Seismic Design Codes," *Proceedings of SMIP92, Seminar on Seismological and Engineering Implications of Recent Strong-Motion Data*, California Division of Mines and Geology, Sacramento, California, pp. 5-1 to 5-10.

Uang, Chia-M., 1993, "An Evaluation of Two-Level Seismic Design Procedure," *Earthquake Spectra*, Earthquake Engineering Research Institute, Oakland, California, Vol. 9, No. 1, pp. 121–135.

Wilson, E. L., Der Kiureghian, A., and Bayo, E. P., 1981, "A Replacement for the SRSS Method," *Earthquake Engineering and Structural Dynamics*, Elsevier Applied Science, New York, New York.

C4. Foundations and Geotechnical Hazards (Systematic Rehabilitation)

C4.1 Scope

The fundamental reason for including consideration of foundations and geotechnical hazards in seismic rehabilitation of existing buildings is to improve the overall performance of the buildings. The geotechnical engineer and engineering geologist should work directly with the structural engineer and the building owner or the owner's representative, when necessary, to achieve the optimum rehabilitation strategy for the desired Rehabilitation Objective.

Typically, foundations have performed reasonably well on sites where ground displacement has not occurred because of surface faulting, landsliding, or liquefaction. Furthermore, modifying foundations to improve their performance during anticipated earthquake loading can be very costly because of the limited working space, as well as the presence of the building. Therefore, it is desirable to undertake costly foundation modifications only when they are essential to meeting seismic Rehabilitation Objectives for the building.

In addition to addressing building foundation capacities and deformations during earthquakes, the guidelines address other potential geologic hazards associated with earthquakes that may affect the performance of buildings on some sites.

C4.2 Site Characterization

In gathering data for site characterization, the following should be included:

- Visual inspection of the structure and its foundation
- Review of geotechnical reports, drawings, test results, and other available documents directly related to the building
- Review of regional or local reports related to geologic and seismic hazards, and subsurface conditions
- Site exploration, including borings and test pits
- Field and laboratory tests

The scope of the documentation program for a building depends upon specific deficiencies and the Rehabilitation Objective. In some cases, the cost of extensive analysis and testing can be justified by producing results that will allow the use of more accurately determined material properties than the conservative default values prescribed by the *Guidelines*.

Geotechnical information will be required to establish the subsurface conditions that exist beneath the building, to describe the building foundations, and to assess potential earthquake-related hazards that may affect the performance of the site. The general procedure for evaluating foundations and geotechnical information is outlined on Figure C4-1. In many instances, existing data may be sufficient to characterize the site. However, a detailed site assessment may be required for:

- Structures that require an enhanced level of seismic performance
- Facilities that are supported upon deep foundations
- Facilities that are located within areas that may be subjected to fault rupture, liquefaction, lateral spreading, differential compaction, and landsliding

Such detailed site assessments may be conducted with existing information or with new subsurface data. The following text discusses data sources that should be reviewed in the site characterization, along with the requirements for defining the subsurface conditions and describing the existing foundations.

Data Sources. Information required to adequately characterize a site will likely be derived from a combination of several sources, including existing data, a site reconnaissance, and site-specific studies. Potential data sources include the following:

- geological maps
- topographical maps
- hazard maps
- geotechnical reports
- design/construction drawings

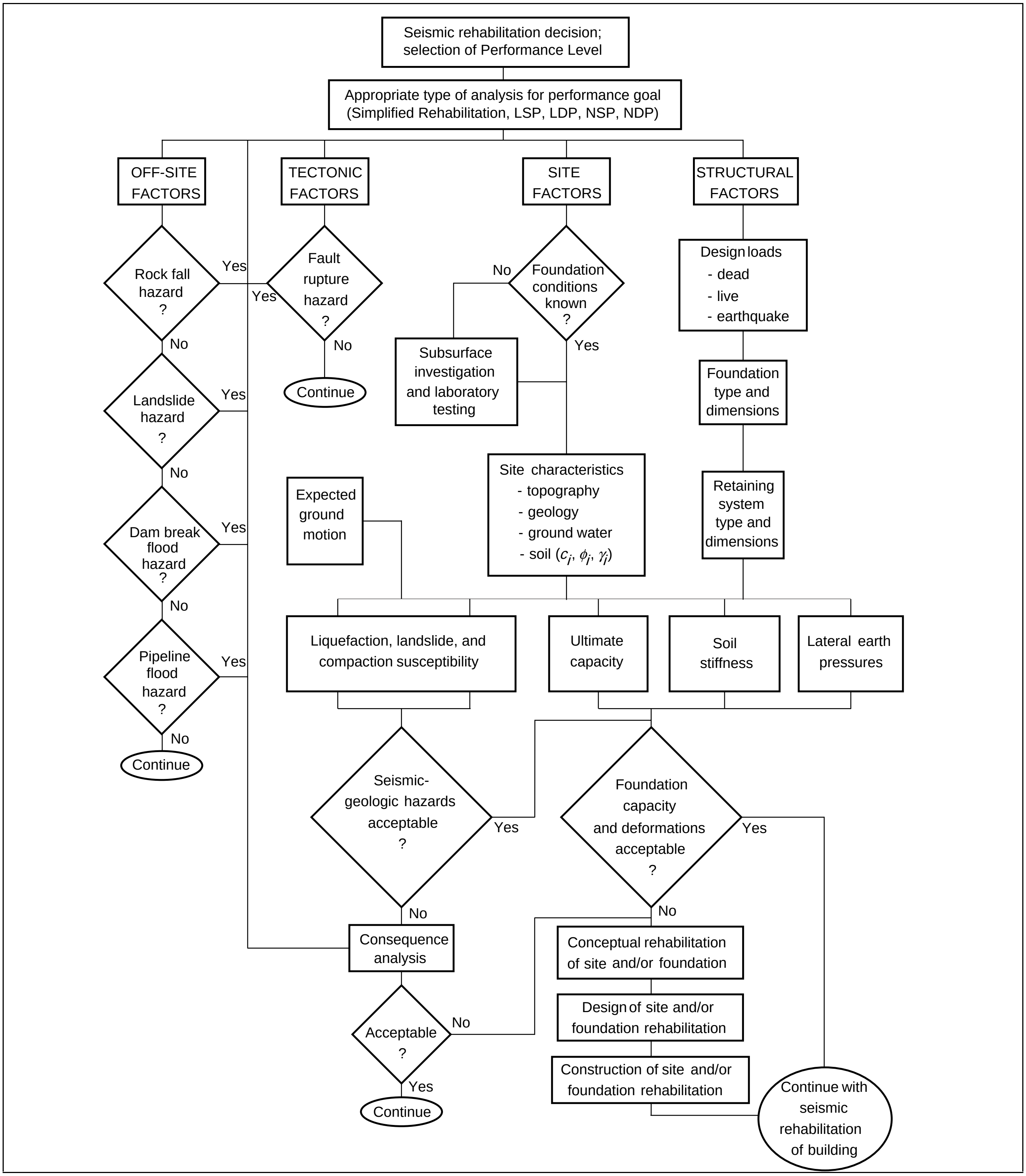


Figure C4-1 General Procedure: Evaluating Foundations and Geotechnical Information

Regional maps—including topographic maps and geologic maps—may be used to provide a general source of information on the conditions in the vicinity of the site. Topographic maps can be useful in assessing the landslide hazard potential that may affect the site. Similarly, geologic maps can provide information on surficial geologic units that may be related to ground stability. Finally, various hazard maps may exist indicating potential earthquake faults, and areas potentially susceptible to liquefaction, landsliding, and flooding or inundation. All of these maps may be used to provide an assessment of the large-scale performance of the site.

On a more local level, site-specific information may be obtained from geotechnical reports and foundation drawings. Relevant site information to be obtained from geotechnical reports includes logs of borings and/or cone penetrometer tests and laboratory tests to determine shear strengths of the subsurface materials, and engineering assessments that may have been conducted addressing geologic hazards, such as faulting, liquefaction, and landsliding. If geotechnical reports are not available for the subject facility, geotechnical reports for adjacent buildings may also provide a basis for developing the engineering assessments of the earthquake performance of the site. Finally, information should be obtained from geological reports or other regional studies regarding potential depths of the groundwater table.

Information contained on existing building drawings should be reviewed for relevant foundation data. This data would include the type, size, and location of all footings and footing design loads.

In addition to gathering existing data, a site reconnaissance should be performed to document the performance of the site and building. The site reconnaissance is conducted to gather information for several purposes. First, the reconnaissance should confirm that the actual site conditions agree with information obtained from the building drawings. Variances from the building drawings should be noted and considered in the evaluation. Such variances include building additions or foundation modifications that are not shown on the existing documentation.

A second purpose is to ascertain the presence of a potentially hazardous condition, such as a nearby steep slope susceptible to landsliding or rock fall, or a stream channel toward which lateral spreading could occur. A third purpose of the site reconnaissance is to document off-site development that may have a potential impact on the building. Such off-site development could include building grading activities that may impose a load or reduce a level of lateral support to the structure under consideration.

The site reconnaissance also should document the performance of the existing building and the adjacent area to denote signs of poor foundation performance, such as settlement of floor slabs, foundations, or sidewalks. These indicators may suggest structural distress that could affect performance during a future earthquake, as well as indicate the presence of soils that might settle during an earthquake.

The existing site data and information gained from the site reconnaissance may need to be supplemented by additional site explorations where there is a significant potential for the site to be affected by fault rupture, liquefaction, lateral spreading, differential compaction, or landsliding, or where the site has exhibited poor performance as reflected in ground settlement or building settlement. Under these conditions, detailed subsurface information will be required to define the subsurface stratigraphy and the engineering properties of the underlying soils. While the scope and extent of such explorations depends upon the number and type of existing studies that have been conducted at the site, new explorations may be required to augment the existing database. Applicable subsurface exploration procedures include:

- exploration borings
- cone penetrometer tests (CPTs)
- seismic cone penetrometer tests (SCPTs)
- standard penetration tests (SPTs)
- test pits
- laboratory testing

Buildings with shallow foundations often can be evaluated adequately by test pits, particularly if footing dimensions or conditions are unknown. Test pits or borings extending 10–15 feet below the footing often provide adequate geotechnical information. End-driven tube samples should be collected from test pit exposures; shoring of test pit walls must be done to provide safety during sampling and to comply with safety regulations.

Buildings with deep foundations may require borings with SPTs, CPTs, and/or SCPTs to provide adequate geotechnical information on the stratigraphy and material properties of the underlying soils. Explorations must extend below the depth of influence of the foundations. This depth, determined by a geotechnical engineer, depends on the foundation type and the nature of the subsurface materials. SPT sampling should be done at frequent intervals (3–5 feet) within the site borings. Undisturbed sampling should be conducted, where possible, within the underlying soil units to provide suitable samples for laboratory testing to determine unit weight, soil shear strengths, and friction angles of the underlying soil. More detailed stratigraphic information can be obtained from CPTs and SCPTs. Soil stiffnesses may be determined directly from the results of the SCPTs, or indirectly through empirical correlations with static soil properties.

If general information about the site region is known well enough to indicate uniform conditions over the dimensions of the building, then one boring, sounding, or test pit may be adequate. However, two or more borings, soundings, test pits, or a combination of the subsurface investigation techniques will be needed to increase confidence that the site is being adequately characterized. The adequate number of subsurface investigation locations depends on the size of the site, the complexity of the site geology, and the importance of the structure.

C4.2.1 Foundation Soil Information

It is necessary to define subsurface conditions at each building location in sufficient detail so as to assess the ultimate capacity of the building foundations and to determine if the site may be potentially affected by an earthquake-related hazard, such as earthquake-induced landsliding, lateral spreading, and liquefaction. The level to which subsurface conditions need to be defined depends on the Rehabilitation Objective for the facility and the specific foundations and subsurface conditions.

As a minimum, the site stratigraphy must be defined to establish the materials that underlie the foundations. This assessment must include information on the material composition (sand/clay) and the consistency or relative density of the underlying soil units. The consistency or the relative density of the underlying soil may be assumed from empirical correlations of SPT N-values. Additionally, the definition of the site subsurface conditions must include an assessment of the location of the water table beneath the structure and any seasonal fluctuations of the water table. Fluctuations of the water table may affect the ultimate bearing capacity of the building foundations and the potential for liquefaction.

With this minimum amount of information, presumptive or prescriptive procedures may be used to determine the ultimate bearing capacity of the foundations. However, additional information is required for site-specific assessments of foundation bearing capacity and stiffness. Acquiring this additional information involves determining unit weights, shear strength, friction angle, compressibility characteristics, soil moduli, and Poisson's ratio.

The site characterization also requires information defining the type, size, and location of the foundation elements supporting the structure. Types of foundations include spread footings, mats, driven pile foundations, cast-in-place piles, and drilled piers. Other required information includes the size of the foundation elements, locations of the base of the footings or the tips of the piles, the pile cap elevations, foundation material composition (i.e., wood, steel, or concrete piles), and pile installation methods (i.e., opened- or closed-end piles, driven or jetted). The design drawings may also indicate information regarding the allowable bearing capacity of the foundation elements. This information can be used directly in a presumptive or prescriptive evaluation of the foundation capacity. Construction records may also be available indicating ultimate pile capacities if load tests were performed. Finally, information on the existing loads on the structure is relevant to determining the amount of overload that the foundations may be capable of resisting during an earthquake.

C4.2.2 Seismic Site Hazards

Earthquake-related site hazards—including fault rupture, liquefaction, differential compaction, landsliding, or flooding—can affect the ability of a structure or building to meet the desired seismic Performance Level. In some instances, the probability of occurrence of these hazards is small enough that they may be neglected, depending on the Rehabilitation Objectives for a specific project.

The *Guidelines* provide information on evaluation of site hazards. An initial assessment for each hazard can be conducted based on readily available data. This initial assessment might result in an indication that further consideration of a specific hazard is unnecessary. For example, on hillside sites with slopes of less than some prescribed value, landsliding need not be a design consideration. If a specific hazard cannot be eliminated from further consideration, the *Commentary* provides resources for more detailed investigations.

The result of the detailed investigation of site hazards will be to predict the nature and magnitude of ground movement for use by a structural engineer in the rehabilitation design. The events causing these movements must be consistent, in a probabilistic sense, with the chosen Performance Levels for the rehabilitation. It makes no sense to rehabilitate a structure to remain operational after a 500-year earthquake if a landslide with a much greater chance of occurrence could cause its collapse.

C4.2.2.1 Fault Rupture

Ground displacements generally are expected to recur along preexisting faults. The development of a new fault or reactivation of a very old (pre-Quaternary) fault is uncommon and generally need not be a concern for typical buildings. In general, the more recent and frequent the displacement is along a fault, the greater the probability of future faulting. The evaluation of future fault-rupture hazards involves careful application of skills and techniques not commonly used in other engineering geologic investigations (e.g., detailed examination of trench exposures and radiometric dating of geologic materials). Many active faults are complex, consisting of multiple breaks that may have originated during different surface-faulting earthquakes. To accurately evaluate the potential hazards of surface fault rupture, the engineering geologist must determine:

- The locations of fault traces
- The nature and amount of near-surface fault deformations (shear displacements and folding or warping)
- The history of the deformations

Key parameters are the age of the most recent displacement and the recurrence interval between successive displacements. Guidelines for evaluating surface fault rupture hazards have been developed in California and Utah (California Division of Mines and Geology, 1975; Slosson, 1984; Utah Section of the Association of Engineering Geologists, 1987). Maps showing the location of faults that have been active during Quaternary time (the most recent 1.8 million years of earth history) have been prepared for a number of regions (e.g., Nakata et al., 1982; Jennings, 1992; Hecker, 1993) and local areas (e.g., Hart et al., 1981; Bell, 1984; Personius and Scott, 1990).

Buildings found to straddle active faults must be assessed to determine if any rehabilitation is warranted—possibly to reduce collapse potential of the structure, given the likely amount and direction of fault displacement. Fault rupture is generally treated differently from seismic hazards related to ground motion. Active faults are considered capable of rupturing the ground surface on the basis of deterministic reasoning. Ground motion and the secondary hazards caused by it (liquefaction and landsliding) are evaluated with probabilistic reasoning. Thus, a site susceptible to liquefaction under ground motion considered to be less likely than 10%/50 years may be judged to have an acceptable risk, and seismic rehabilitation may proceed. However, a site straddling a fault considered to have displaced the ground surface two feet during the past 10,000 years may be judged to have an unacceptable risk, and rehabilitation may be abandoned. It is generally considered unacceptable for a new building to be situated straddling the trace of an active fault. However, policy has yet to be developed regarding the value and utility of an existing building that straddles an active fault.

Active faults differ in degree of activity and amount and character of displacement. Major active faults exhibit large amounts of displacement, which can be concentrated on a single trace, or several relatively closely spaced traces. Minor active faults exhibit small amounts of displacement on individual traces and can have a moderate amount of displacement distributed across an area. Active faults have caused strike-slip, normal-slip, and reverse-slip displacement (Figure C4-2a, b, c, respectively). Examples are the 1992 Landers earthquake in California, the 1983 Borah Peak earthquake in Idaho, and the 1971 San Fernando earthquake in California, respectively. In some geologic environments, surface fault rupture is oblique-slip (strike plus normal or reverse). Active faults commonly display a variety of characteristic landforms attesting to geologically youthful displacements. Figure C4-3

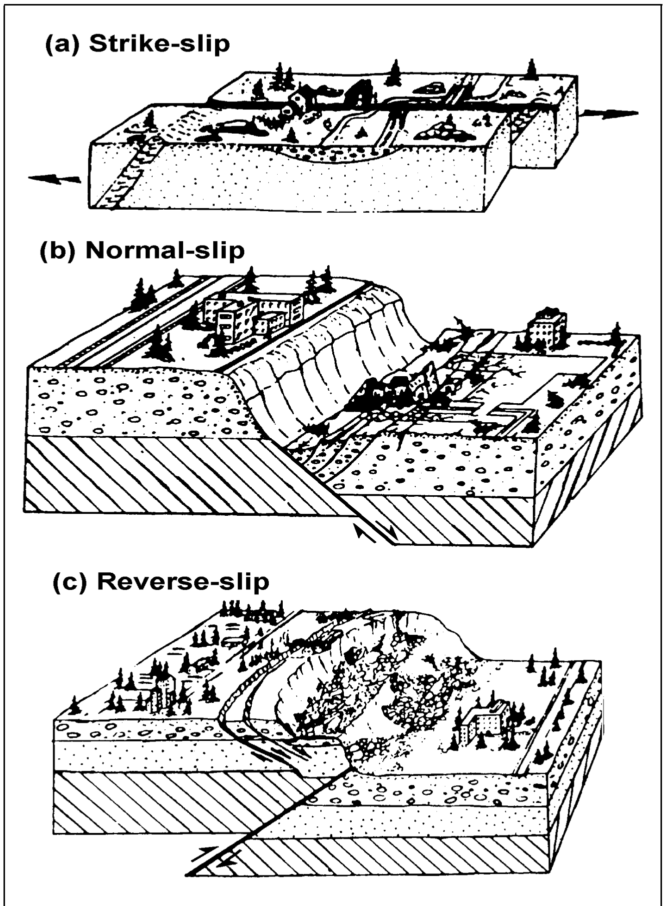


Figure C4-2 Schematic Diagrams of Surface Fault Displacement (modified from Slemmons, 1977)

illustrates some geomorphic features along active strike-slip faults.

C4.2.2.2 Liquefaction

Soil liquefaction is a phenomenon in which a soil below the groundwater table loses a substantial amount of strength due to strong earthquake ground shaking. Recently deposited (i.e., geologically young) and relatively loose natural soils and uncompacted or poorly compacted fill soils are potentially susceptible to liquefaction. Loose sands and silty sands are particularly susceptible; loose silts and gravels also have potential for liquefaction. Dense natural soils and well-compacted fills have low susceptibility to liquefaction. Clay soils are generally not susceptible, except for highly sensitive clays found in some geographic regions.

The *Guidelines* provide criteria that facilitate screening sites that do not have a significant liquefaction hazard. In addition to these criteria, if the site is located in an area where a regional mapping of liquefaction potential has been carried out by the USGS or other governmental agency, then such mapping might also be used to screen for a liquefaction hazard. Generally, sites located in areas characterized as having a low or very low liquefaction hazard can be screened out. However, definitions used in regional liquefaction potential zonations vary, and the definitions, bases, uncertainty, and qualifications associated with the zonation should be carefully reviewed before relying on regional maps.

The following paragraphs provide guidelines for evaluating liquefaction potential for cases where the hazard cannot be screened out. The occurrence of liquefaction by itself does not necessarily imply adverse consequences to a structure. Potential consequences of liquefaction include lateral spreading and flow slides, bearing capacity failure, settlements, increased lateral pressures on retaining walls, and flotation of buried structures. It is essential to assess the consequences of liquefaction and their effects on the structure. Thus, guidelines for such assessment are also presented below. Measures that may be considered to mitigate liquefaction hazards are discussed in Section C4.3.2.

In assessing liquefaction potential, available geotechnical data on the local geology (particularly the age of the geologic units) and the subsurface soil and groundwater conditions should be examined. Often, sufficient data are available from prior geotechnical investigations. If not, supplemental borings can be made or other subsurface investigation techniques (e.g., CPTs) can be used. Simplified, empirically-based procedures using blow count data from soil borings (or CPT data) generally can be used to evaluate liquefaction susceptibility. Occasionally, when dealing with soil types for which empirical correlations are less applicable, such as silts and gravels, it may be necessary to conduct special field and/or laboratory investigations.

Seed-Idriss Procedure for Evaluating Liquefaction Potential. The potential for liquefaction to occur may be assessed by a variety of available approaches (National Research Council, 1985). The most commonly utilized approach is the Seed-Idriss simplified empirical procedure—presented by Seed and Idriss (1971, 1982) and updated by Seed et al. (1985) and Seed and Harder (1990)—that utilizes SPT blow

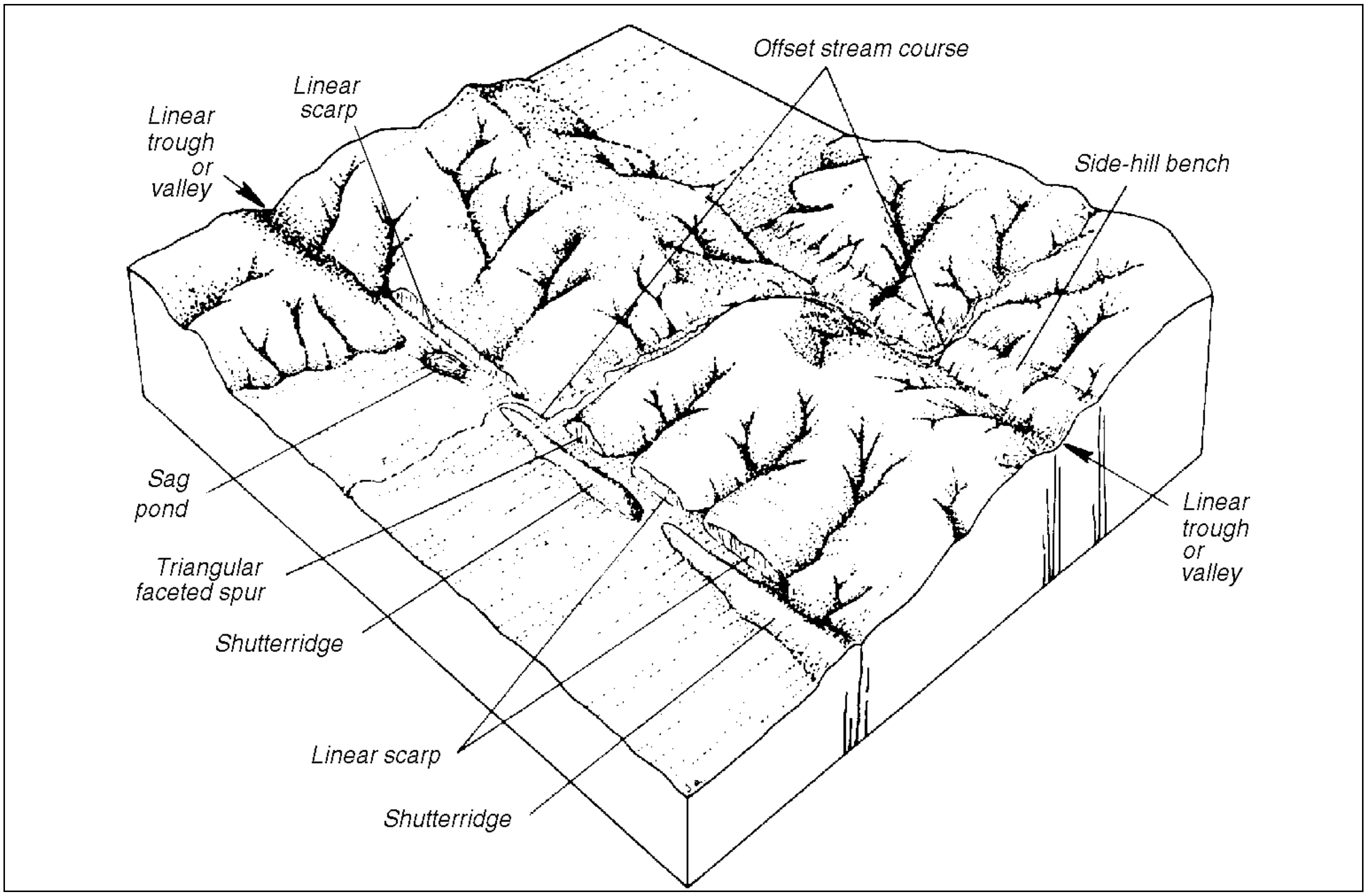


Figure C4-3 Features Commonly Found along Active Strike-Slip Faults (modified from Slemmons, 1977)

count data. Using SPT data to assess liquefaction potential due to an earthquake is considered a reasonable engineering approach (Seed and Idriss, 1982; Seed et al., 1985; National Research Council, 1985), because many of the factors affecting penetration resistance affect the liquefaction resistance of sandy soils in a similar way, and because these liquefaction potential evaluation procedures are based on actual performance of soil deposits during worldwide historical earthquakes.

The basic correlation used in the Seed-Idriss evaluation procedure is shown in Figure C4-4. The plot relates the cyclic stress ratio, τ_{av}/σ'_o, required to cause liquefaction to the normalized blow count obtained from SPT measurements in soil borings. In Figure C4-4, $(N_1)_{60}$ refers to SPT blow count values obtained using a standard 60% hammer energy efficiency and normalized to an effective overburden pressure of 2 ksf. Seed and Idriss (1982) and Seed et al. (1985) provide procedures to convert actual SPT blow counts measured in soil borings to $(N_1)_{60}$ values. Using the simplified procedure of Seed and Idriss (1971), values of τ_{av}/σ'_o induced in the soils by the earthquake ground shaking can be calculated and compared with the values of τ_{av}/σ'_o required to cause liquefaction as determined by the site measurements $(N_1)_{60}$ and Figure C4-4. The simplified procedure equation for calculating the induced cyclic stress ratio is:

$$\frac{\tau_{av}}{\sigma'_o} = 0.65\frac{PGA}{g}\frac{\sigma_o}{\sigma'_o}r_d \qquad \text{(C4-1)}$$

where

τ_{av}/σ'_o = Induced cyclic stress ratio

PGA = Peak ground acceleration (g units)

σ_o = Total overburden pressure at a depth of interest

σ'_o = Effective overburden pressure at a depth of interest

r_d = Stress reduction factor that decreases from a value of 1.0 at the ground surface to a value of 0.9 at a depth of about 35 feet

As an alternative to comparing the induced cyclic stress ratios with those required to cause liquefaction, critical values of $(N_1)_{60}$ can be determined from Figure C4-4 for the induced cyclic stress ratios obtained using Equation C4-1; these critical $(N_1)_{60}$ values can then be compared with the actual $(N_1)_{60}$ values for the site. For

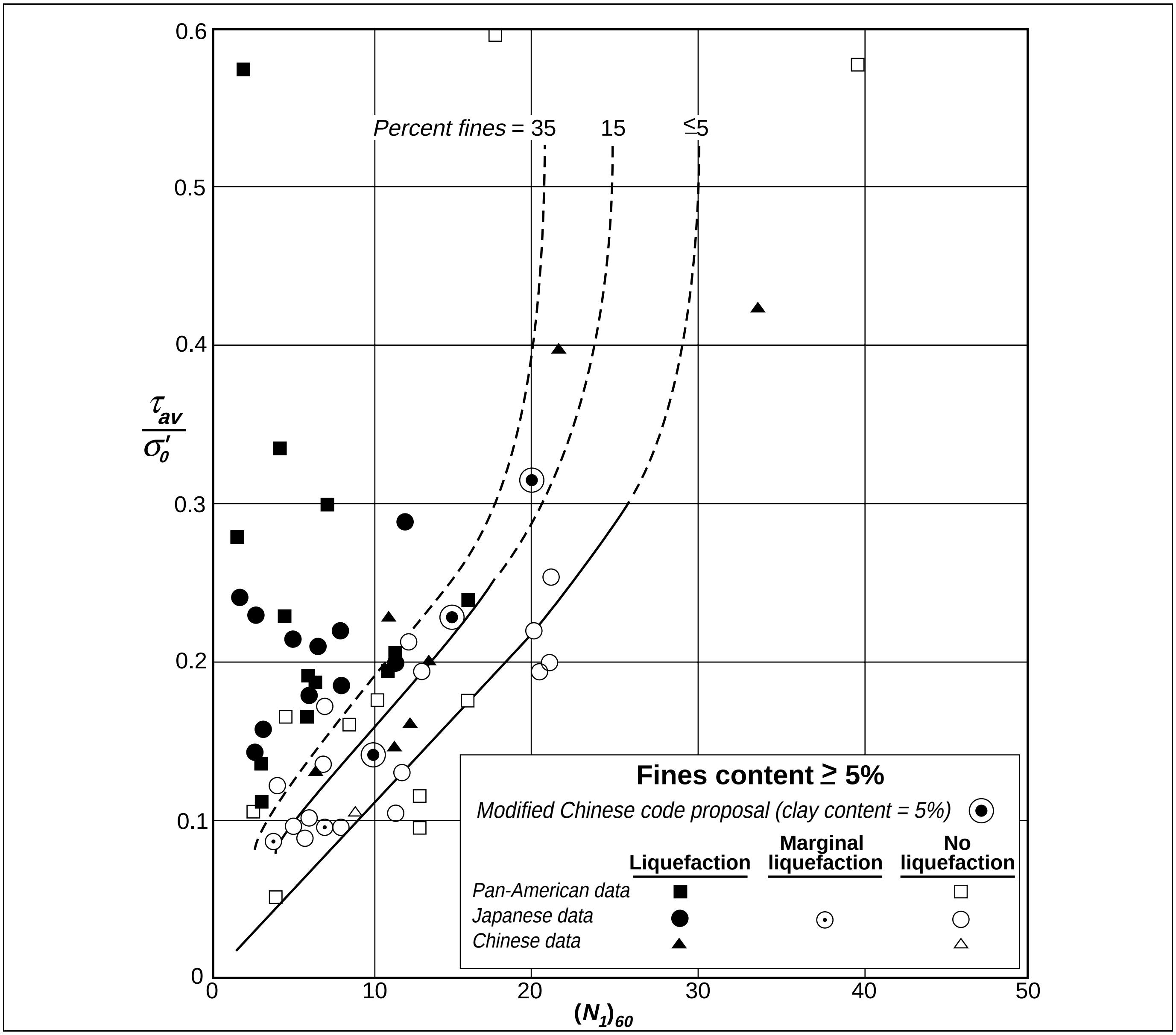


Figure C4-4 ***Relationship Between Cyclic Stress Ratio Causing Liquefaction and $(N_1)_{60}$ values for M = 7.5 Earthquakes (from Seed et al., 1985)***

example, Figure C4-5 illustrates a comparison between the critical $(N_1)_{60}$ line obtained based on the site peak ground acceleration and Figure C4-4, and the actual $(N_1)_{60}$ data for a site. In this illustration, the critical

$(N_1)_{60}$ line exceeds most of the site $(N_1)_{60}$ values, indicating liquefaction is likely to occur in this case. It should be recognized that the Seed-Idriss simplified procedure is based on average $(N_1)_{60}$ values at site; Fear and McRoberts (1995) conducted a reinterpretation of the catalogue of case histories that provided the basis for the Seed-Idriss simplified procedure systematically using minimum $(N_1)_{60}$, and pointed out the excess conservatism that could arise from treating the $(N_1)_{60}$ values from the Seed-Idriss curves as representing threshold (minimum) values.

CPT data may also be utilized with the Seed-Idriss approach by conversion to equivalent SPT blow counts, using correlations developed among cone tip resistance Q_c, friction ratio, soil type, and Q_c/N in which N is the SPT blow count (Seed and DeAlba, 1986; Robertson and Campanella, 1985). Direct correlations of CPT data with liquefaction potential have also been developed (Robertson and Campanella, 1985; Mitchell and Tseng, 1990; Robertson et al., 1992), but to date these are not as widely used as the Seed-Idriss correlation with $(N_1)_{60}$ blow count as shown in Figure C4-4.

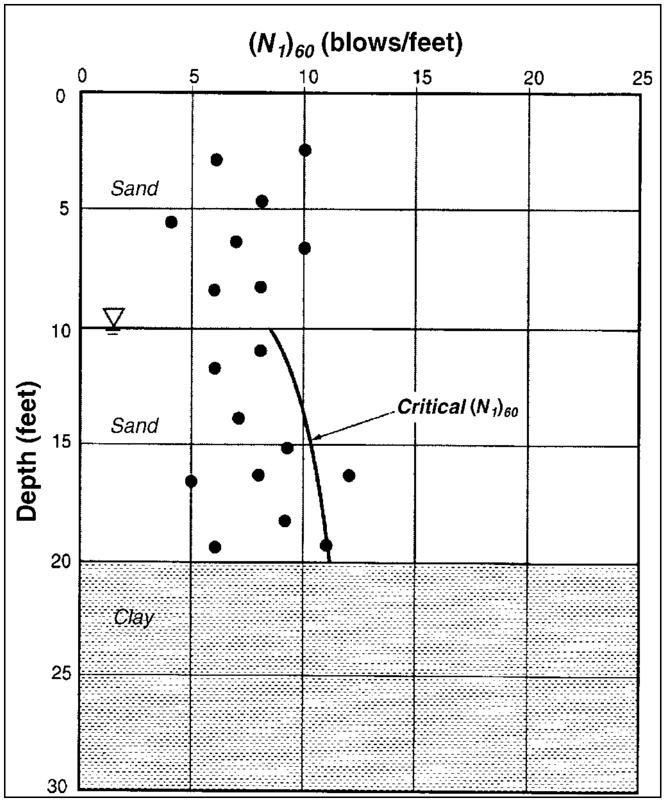


Figure C4-5 Comparing Site $(N_1)_{60}$ Data from Standard Penetration Tests with Critical $(N_1)_{60}$ Values Calculated using the Seed-Idriss Procedure

Evaluating Potential for Lateral Spreading. Lateral spreads are ground-failure phenomena that can occur on gently sloping ground underlain by liquefied soil. Earthquake ground-shaking affects the stability of sloping ground containing liquefiable materials by seismic inertia forces within the slope and by shaking-induced strength reductions in the liquefiable materials. Temporary instability due to seismic inertia forces is manifested by lateral "downslope" movement that can potentially involve large land areas. For the duration of ground shaking associated with moderate to large earthquakes, there could be many such occurrences of temporary instability, producing an accumulation of "downslope" movement. The resulting movements can range from a few inches or less to tens of feet, and are characterized by breaking up of the ground and horizontal and vertical offsets. A schematic of lateral spreading is illustrated in Figure C4-6.

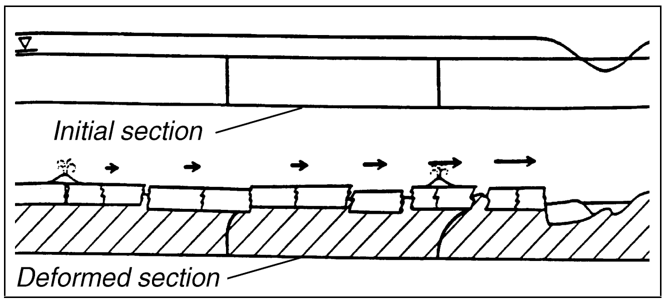


Figure C4-6 Lateral Spread Before and After Failure (from Youd, 1984)

Various relationships for estimating lateral spreading displacement have been proposed, including the Liquefaction Severity Index (LSI) by Youd and Perkins (1978), a relationship incorporating slope and liquefied soil thickness by Hamada et al. (1986), a modified LSI approach presented by Baziar et al. (1992), and a relationship by Bartlett and Youd (1992), in which they characterize displacement potential as a function of earthquake and local site characteristics (e.g., slope, liquefaction thickness, and grain size distribution). The relationship of Bartlett and Youd (1992), which is empirically based on analysis of case histories where lateral spreading did and did not occur, is relatively widely used, especially for initial assessments of the hazard. More site-specific analyses can also be made based on slope stability and deformation analysis procedures using undrained residual strengths for liquefied sand (Seed and Harder, 1990; Stark and Mesri, 1992), along with either Newmark-type simplified displacement analyses (Newmark, 1965; Franklin and Chang, 1977; Makdisi and Seed, 1978; Yegian et al., 1991) or more complex deformation analysis approaches.

Evaluating Potential for Flow Slides. Flow generally occurs in liquefied materials found on steeper slopes and may involve ground movements of hundreds of feet or more. As a result, flow slides can be the most catastrophic of the liquefaction-related ground-failure phenomena. Fortunately, flow slides occur much less commonly than lateral spreads. Whereas lateral spreading requires earthquake inertia forces to create instability for movement to occur, flow movements occur when the gravitational forces acting on a ground slope exceed the strength of the liquefied materials within the slope. The potential for flow sliding can be assessed by carrying out static slope stability analyses using undrained residual strengths for the liquefied materials.

Evaluating Potential for Bearing Capacity Failure. The occurrence of liquefaction in soils supporting foundations can result in bearing capacity failures and large plunging-type settlements. In fact, the buildup of pore water pressures in a soil to less than a complete liquefaction condition will still reduce soil strength and may threaten bearing capacity if the strength is reduced sufficiently. Figure C4-7 illustrates how excess pore water pressures relate to the factor of safety against liquefaction, where the factor of safety is the stress ratio required to cause liquefaction (for example, from Figure C4-4) divided by the stress ratio induced in the soils by the earthquake ground shaking. If the factor of safety is less than about 1.5, excess pore pressure development may become significant. The amount of excess pore water pressure development may be evaluated using data such as shown in Figure C4-7.

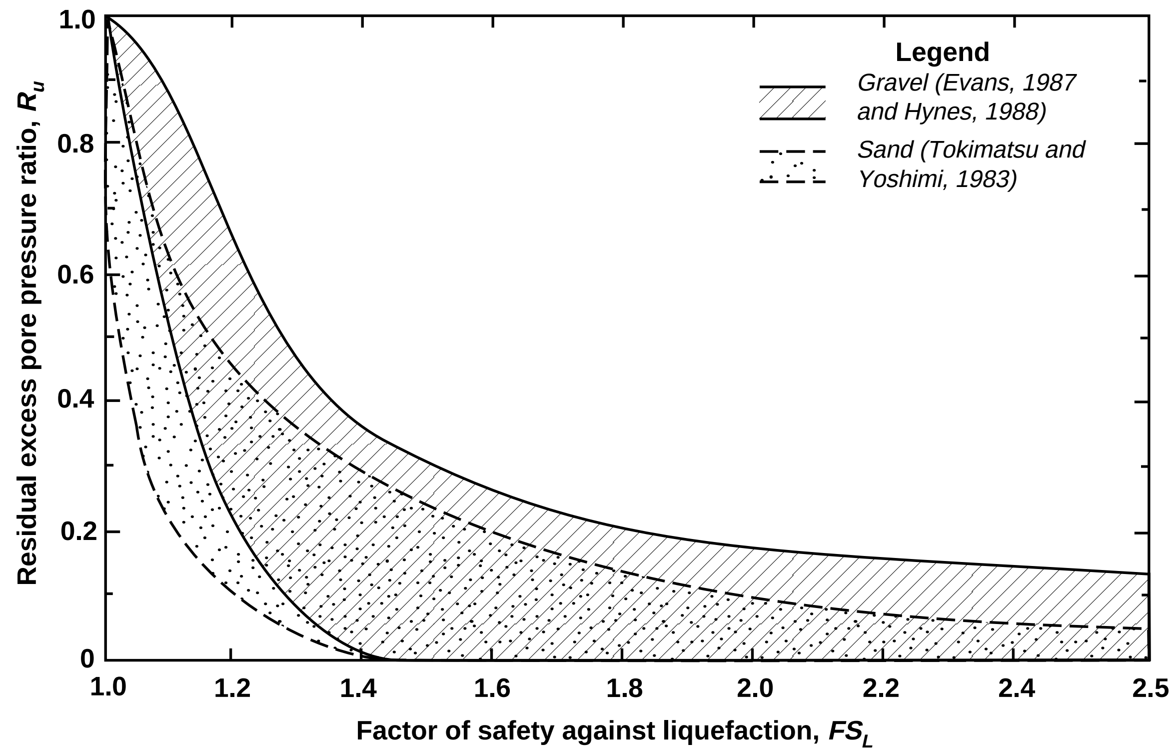


Figure C4-7 ***Typical Relationships for Sand and Gravel (from Marcuson and Hynes, 1990)***

The potential for bearing capacity failure beneath a spread footing depends on the depth of the liquefied (or partially liquefied) layer below the footing, the size of the footing, and the load. If lightly-loaded small footings are located sufficiently above the depth of liquefied materials, bearing capacity failure may not occur. The foundation bearing capacity for a case where a footing is located some distance above a liquefied layer can be assessed by evaluating the strength of the liquefied (excess pore pressure ratio = 1.0), partially liquefied (excess pore pressure ratio <1.0, Figure C4-7), and nonliquefied strata, then applying bearing capacity formulations for layered systems (Meyerhof, 1974; Hanna and Meyerhof, 1980; Hanna, 1981). The capacity of friction pile or pier foundations can be similarly assessed, based on the strengths of the liquefied, partially liquefied, and nonliquefied strata penetrated by the foundations.

Evaluating Potential for Liquefaction-Induced Settlements. Following the occurrence of liquefaction, over time the excess pore water pressures built up in the soil will dissipate, drainage will occur, and the soil will densify, manifesting at the ground surface as settlement. Differential settlements occur due to lateral variations in soil stratigraphy and density. Typically, such settlements are much smaller and tend to be more uniform than those due to bearing capacity failure. They may range from a few inches to a few feet at the most where thick, loose soil deposits liquefy.

One approach to estimating the magnitude of such ground settlement, analogous to the Seed-Idriss simplified empirical procedure for liquefaction potential evaluation (i.e., using SPT blow count data and cyclic stress ratio), has been presented by Tokimatsu and Seed (1987); the relationships they presented are shown on Figure C4-8. Relationships presented by Ishihara and Yoshimine (1992) are also available for assessing settlement.

Evaluating Increased Lateral Earth Pressures on Retaining Walls. Behind a retaining wall, the buildup of pore water pressures during the liquefaction process increases the pressure on the wall. This pressure is a static pressure, which reduces with time after the earthquake as pore pressures dissipate. The increased lateral pressures due to either partial or complete liquefaction of the backfill are readily calculated using conventional static earth pressure formulations. For the case of complete liquefaction, the total earth pressures are those of a fluid having a unit weight equal to the total unit weight of the soil.

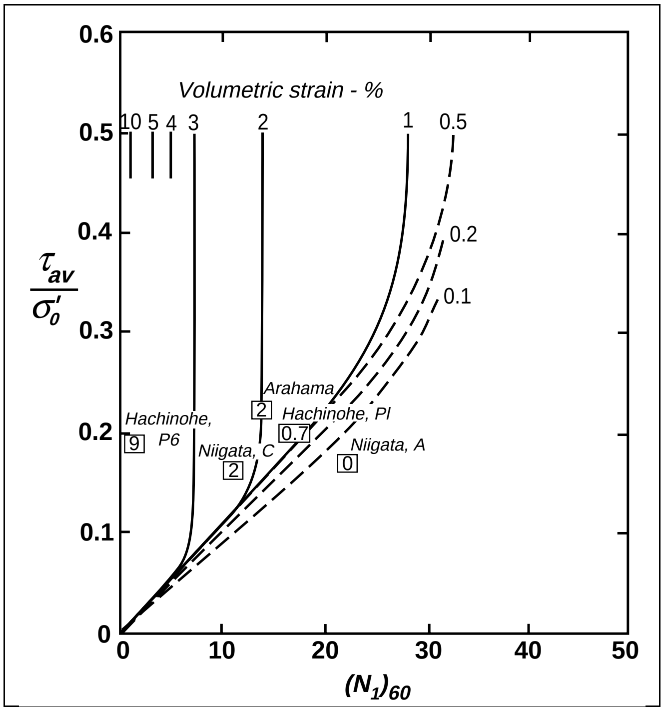


Figure C4-8 ***Relationship among Cyclic Stress Ratio, $(N_1)_{60}$, and Volumetric Strain for Saturated Clean Sands (from Tokimatsu and Seed, 1987)***

Evaluating Potential for Flotation of Buried Structures. A common phenomenon accompanying liquefaction is the flotation of tanks or structures that are embedded in liquefied soil. A building with a basement surrounded by liquefied soil can be susceptible to either flotation or bearing capacity failure, depending on the building weight and the structural continuity (i.e., whether the basement acts as an integral unit). The potential for flotation of a buried or embedded structure can be evaluated by comparing the total weight of the buried or embedded structure with the increased uplift forces occurring due to the buildup of liquefaction-induced pore water pressures.

C4.2.2.3 Differential Compaction

A procedure to evaluate settlement associated with post-liquefaction densification of soils below the water table was just discussed in Section C4.2.2.2. Loose cohesionless soils above the water table will also tend to densify during the period of earthquake ground shaking, as the earthquake-induced shear strains cause the soil particles to shift into a denser state of packing. Procedures described by Tokimatsu and Seed (1987) may be used to estimate settlements of cohesionless soils above the groundwater table. The simplified procedures they described may be used to estimate the shear strains induced by the ground shaking. The graph in Figure C4-9, which is based on laboratory unidirectional cyclic tests, may then be used to estimate volumetric strains (percent settlements) as a function of the induced shear strains and the normalized SPT blow counts of the soils. The graph in Figure C4-9 is for 15 cycles of shaking corresponding to a magnitude 7.5 earthquake; Tokimatsu and Seed provide scaling factors for other magnitude earthquakes. The graph is also for one horizontal component of ground motion. As described by Tokimatsu and Seed (1987), research by Pyke et al. (1975) indicated that volumetric strains due to multidirectional shaking are about twice those due to unidirectional shaking. Therefore, the settlement obtained using Figure C4-9 should be doubled to estimate field settlements.

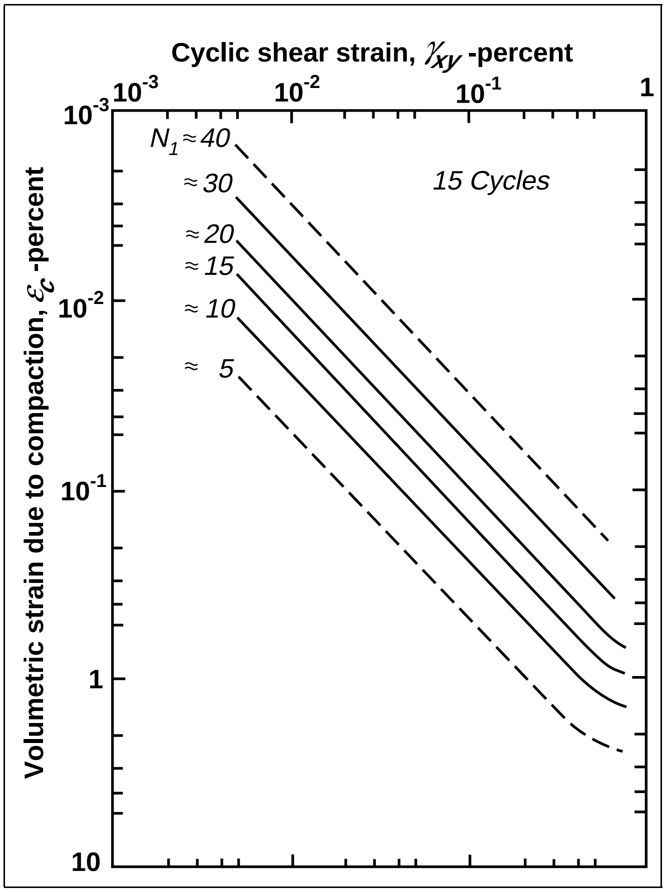


Figure C4-9 Correlation for Volumetric Strain, Shear Strains, and $(N_1)_{60}$ (from Tokimatsu and Seed, 1987)

Situations most susceptible to differential compaction include heavily graded areas where deep fills have been placed to create building sites for development. If the fills are not well compacted, they may be susceptible to significant settlements, and differential settlements may occur above variable depths of fill placed in canyons and near the transitions of cut and filled areas.

C4.2.2.4 Landsliding

Earthquake-induced landslides represent a significant hazard to the seismic performance of facilities located on steep slopes in marginally stable areas. Landslides may affect a structure by directly undermining a facility, resulting in structural damage. Alternatively, off-site landslides could develop above a structure, and the debris from the landslide (avalanche, rock fall, or debris torrent) could impinge upon a structure and lead to undesirable performance. Thus, consideration of landslide effects should include both on-site and off-site sources. Sites that are more likely to be affected by earthquake-induced landslides include locations with slopes of 18 degrees or greater, or a history of rock falls, avalanches, or debris torrents.

Stability analysis shall be performed for all sites located on slopes steeper than three horizontal to one vertical (approximately 18 degrees), and the stability analysis should consider the following factors:

- Slope geometry
  - slope inclination
  - slope height
- Subsurface conditions
  - stratigraphy (material type and bedding)
  - material properties (unit weight, friction angle, and cohesion)
  - groundwater conditions (level, perched locations, and hydrostatic pressures)
- Level of ground shaking

Pseudo static analyses may be used to evaluate landsliding potential. Such analyses should be used only in instances where liquefaction would not develop and where the underlying materials would not suffer major strength degradation as a result of earthquake ground shaking (i.e., soft, sensitive clays). The analyses should be conducted using a seismic coefficient equal to one-half the peak ground acceleration for the site area. A safety factor of at least 1.0 should be obtained. The pseudo-static analysis is conservative because it is performed with a continuously applied horizontal force acting in the downhill direction. A static factor of safety of 1.0 is considered acceptable for this type of analysis. Safety factors of 1.5 are appropriate for static vertical load conditions, which the slopes must meet independently.

If the results from the pseudo-static analyses indicate a safety factor of less than 1.0, sliding block analyses such as Newmark's (1965) method should be conducted. The Newmark analyses may consider the potential effects of both on-site and off-site stability. The advantage of the Newmark procedure is that it provides an evaluation of the permanent ground deformation that may occur as a result of earthquake ground shaking. This evaluation of deformation may be used in developing structural strengthening to withstand this level of deformation (see Sections 4.3 and 4.6).

Earthquake-induced rock fall hazards exist only if a cliff or steep slope with blocks available to fall is located in close proximity upslope from the building site. Where this is the case, blocks of rock often fall from such cliffs or slopes without earthquake shaking, and boulders (often used for landscaping) commonly are present on the site and in the immediate vicinity. Falling rock starts from an at-rest condition, achieves a maximum velocity, and comes to rest again. Blocks of rock that have come to rest beyond the site indicate that such rocks had kinetic energy as they passed over the building site. The amount of energy at the building site can be estimated with the aid of the Colorado Rock Fall Simulation Program (Pfeiffer and Higgins, 1991).

If no blocks of rock are present at the site, but a cliff or steep slope is located nearby, then the likely performance of the cliff under earthquake loading should be evaluated. The earthquake loading condition for cliff performance must be compatible with the earthquake loading condition selected for the Rehabilitation Objective for the building.

Some sites may be exposed to hazards from major landslides moving onto the site from upslope, or retrogressive removal of support from downslope. Such conditions should be identified during site characterization, and may pose special challenges if adequate investigation requires access to adjacent property.

C4.2.2.5 Flooding or Inundation

Flooding hazards originating off-site may adversely affect a building being considered for seismic rehabilitation. Tsunami and seiche can be triggered by earthquakes, causing wave impact and inundation damage at building sites located near shorelines. Failure of reservoirs, aqueducts, and canals upslope from building sites can cause site flooding.

Some buildings may be located in potential flood paths in the event that a dam or pipeline fails during an earthquake. Individual states are responsible for dam safety inspections, and specific information should be available for all high-hazard dams. Pipeline rupture and resulting flood or severe erosion typically has not been addressed. Given the cost of rehabilitation, it may be prudent to consider the consequences of such hazards under earthquake loading compatible with the desired Performance Level for the building.

In low-lying coastal areas, tsunami or seiche processes can be significant for buildings meeting Life Safety or Immediate Occupancy Performance Levels. Historical records of wave run-up should be reviewed, or coastal engineering evaluations of potential wave run-up should be performed as a guide. The return period of the tsunami or seiche should be the same as the earthquake ground motion that serves as the basis for building rehabilitation.

C4.3 Mitigation of Seismic Site Hazards

C4.3.1 Fault Rupture

No commentary is provided for this section.

C4.3.2 Liquefaction

Figure C4-10 illustrates conceptual schemes to mitigate the hazard of liquefaction-induced bearing capacity reduction or settlements due to liquefaction-induced soil densification beneath a building. As stated in the *Guidelines*, the schemes fall into three different categories—modify either the structure, the foundation, or the soil conditions. Figure C4-11 illustrates conceptual schemes to resist liquefaction-induced lateral spreading. The soil may be stabilized beneath the building and, if needed, sufficiently beyond the buildings that liquefaction and spreading of the surrounding areas will not cause significant spreading beneath the building, as illustrated by the stabilized "soil island" concept in Figure C4-11A. Alternatively, a buttress of stabilized ground can be constructed beyond the building to prevent significant lateral spreading behind the buttress, as illustrated in Figure C4-11B. The buttress approach does not prevent settlement from occurring beneath the building, but if bearing capacity failures are not expected (due to lightly loaded footings a sufficient distance above the liquefied zone) and densification settlements are tolerable for the structure (considering the Rehabilitation Objective), then the buttressing approach, by eliminating potentially large spreading-type movements beneath the structure, may be effective.

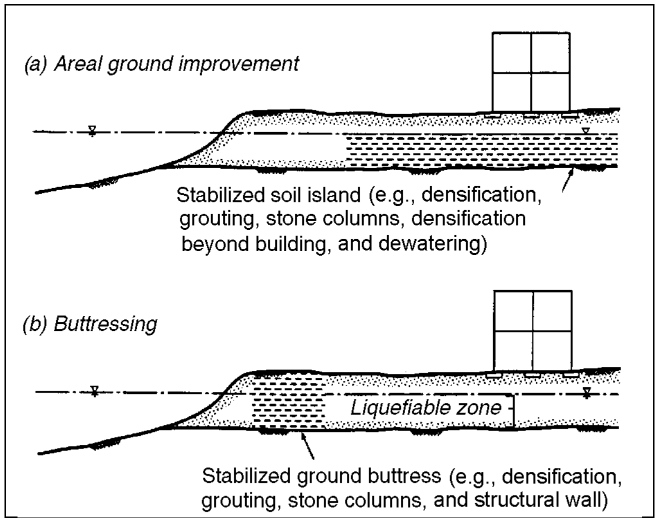


Figure C4-11 Conceptual Schemes to Resist Liquefaction-Induced Lateral Spreading

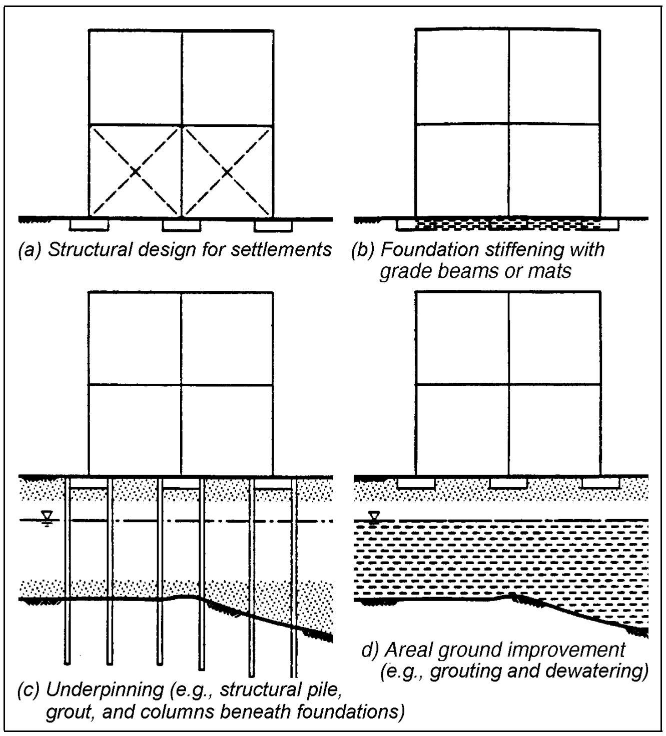


Figure C4-10 Conceptual Schemes to Resist Liquefaction-Induced Settlement or Bearing Capacity Reductions

Ground improvement techniques that can be considered to be used beneath an existing structure include soil grouting, installation of drains, and installation of permanent dewatering systems. In general, ground modification techniques that involve vibratory densification of soils to reduce their liquefaction potential (e.g., vibrocompaction or vibroreplacement) cannot be implemented beneath existing buildings because of the settlements induced during the process.

Different types of grouting are illustrated schematically in Figure C4-12. Compaction grouting, permeation grouting, and jet grouting may have application for mitigation of liquefaction hazard beneath an existing building.

Compaction grouting involves pumping a mixture of soil, cement, and water into the ground to form bulbs of grouted material. The formation of these bulbs compresses and densifies the surrounding soil and increases the lateral earth stresses, thus reducing its liquefaction potential. Effects may be somewhat nonuniform, depending on the spatial pattern of grout bulb formation. The amount of densification that can be achieved may be limited because static compression is less effective than vibration in densifying sands. Compaction grouting must be done carefully to avoid creating unacceptable heaving or lateral displacements during the grouting process.

Permeation grouting involves injecting chemical grout into liquefiable sands to essentially replace the pore water and create a nonliquefiable solid material in the grouted zone. The more fine-grained and silty the sands, the less effective is permeation grouting. If soils are suitable for permeation grouting, this technique can potentially eliminate liquefaction potential.

Jet grouting is a technique in which high-velocity jets cut and mix a stabilizing material such as cement into the soil.

In addition to their use to stabilize entire volumes of soil beneath a building, these grouting techniques can also be used locally beneath individual footings to form stabilized columns of soil, which will transfer vertical foundation loads to deeper nonliquefiable strata.

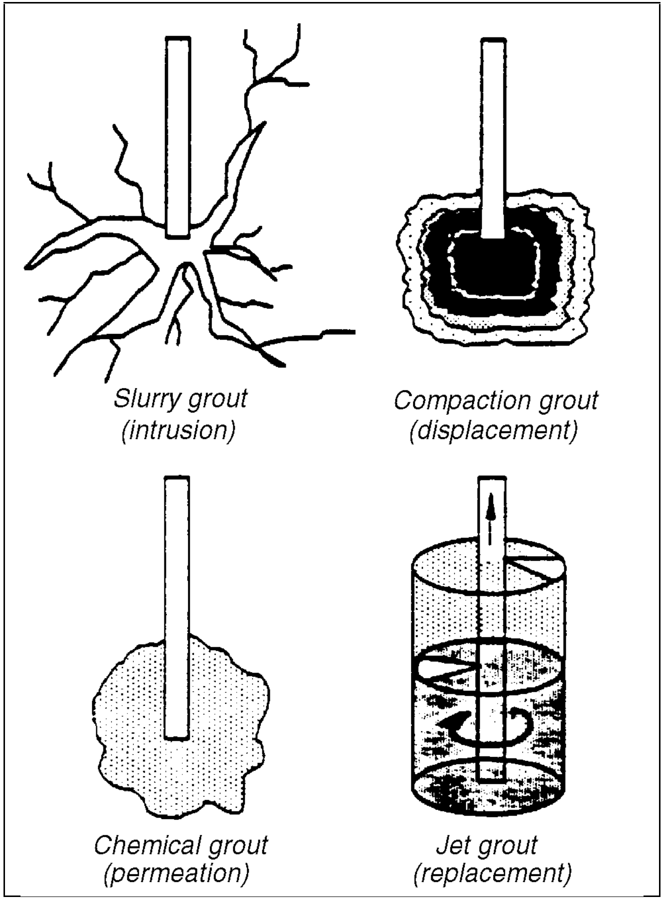


Figure C4-12 Schematic Diagram of Types of Grouting (from notes taken during a 1989 GKN Hayward Baker, Inc., Ground Modification Seminar)

Drain installation (e.g., stone or gravel columns) involves creating closely spaced vertical columns of permeable material in the liquefiable soil strata. Their purpose is to dissipate soil pore water pressures as they build up during the earthquake shaking, thus preventing liquefaction from occurring.

Permanent dewatering systems lower groundwater levels below liquefiable soil strata, thus preventing liquefaction. Because lowering the water table increases the effective stresses in the soil, the potential for causing consolidation in any underlying compressible soil deposits should be evaluated when considering permanent dewatering systems. The dewatering process may also cause settlements in the liquefiable deposits, although in sands these would tend to be small. This alternative also involves an ongoing cost for operating the dewatering system.

Ground stabilization methodologies are discussed in a number of publications, including Mitchell (1981), Ledbetter (1985), National Research Council (1985), Mitchell et al. (1990), and Mitchell (1991). Additional information on these techniques is also available from contractors who specialize in ground modification.

C4.3.3 Differential Compaction

The conceptual mitigation schemes and techniques discussed in Section C4.3.2 can be considered for mitigating the hazard of differential compaction caused by either liquefaction or densification of loose soils above the water table.

C4.3.4 Landslide

The stability of hillside slopes may be improved using a variety of schemes. These range from grading, drainage, buttressing, and soil improvement to structural schemes—retaining walls (gravity, tieback, soil nail, mechanically stabilized earth), barriers, and building options such as grade beams and shear walls. Selection of an appropriate remediation scheme depends on the desired Performance Level for the facility, the size of the potential landslide, and the costs and consequences associated with the earthquake-induced ground movement. Mitigation schemes should be evaluated for acceptable performance using both pseudo-static and dynamic analysis techniques.

C4.3.5 Flooding or Inundation

No commentary is provided for this section.

C4.4 Foundation Strength and Stiffness

The *Guidelines* utilize a stiffness and ultimate capacity approach to evaluating the adequacy of foundations and structures to withstand the imposed static plus seismic loads. In general, soils have considerable ductility unless they degrade significantly in stiffness and strength under cyclic action or large deformations. Degrading soils include cohesionless soils that are predicted to liquefy or build up large pore pressures, and sensitive clays that may lose considerable strength when subject to large strains. Soils not subject to significant degradation will continue to mobilize load, but with increasing deformations after reaching ultimate soil capacity.

The amount of acceptable deformations for foundations in such soils depends primarily on the effect of the deformation on the structure, which in turn depends on the desired Structural Performance Level. However, it should be recognized that foundation yield associated with mobilization at ultimate capacity during earthquake loading may be accompanied by progressive permanent foundation settlement during continued cyclic loading, albeit in most cases this settlement probably would be less than a few inches. In general, if the real loads transmitted to the foundation during earthquake loading do not exceed ultimate soil capacities, it can be assumed that foundation deformations will be relatively small.

If calculated foundation loads exceed twice ($m = 2.0$) the ultimate foundation capacities, two alternatives for evaluating the effects on structural behavior are presented. One alternative is to perform the NSP or NDP, because the nonlinear load-deformation characteristics of the foundations can be directly incorporated in these analyses (Section 4.4.2). Parametric analyses to cover uncertainties in the load-deformation characteristics are recommended. In the static analysis, a somewhat conservative interpretation of the results is recommended because cyclic loading effects cannot be directly incorporated.

For the alternative of a linear procedure using linear foundation springs, wide parametric variations in spring stiffnesses are recommended because of additional uncertainties associated with the linearization of the foundation behavior. This approach is not recommended for the Immediate Occupancy Performance Level.

One of the major changes in traditional seismic design procedures in the *Guidelines* is the direct inclusion of geotechnical and foundation material properties in the Analysis Procedures. In order to accomplish this improvement, the engineer must quantify foundation capacity, stiffness, and displacement characteristics. Considering the multitude of foundation types and soils materials that may be encountered, the authors have concentrated on techniques that may be adapted by qualified experts to generate information for specific projects. For example, a classical general expression for soil bearing capacity is:

$$Q_c = cN_c\zeta_c + \gamma DN_q\zeta_q + \frac{1}{2}\gamma BN_\gamma\zeta_\gamma \qquad \text{(C4-2)}$$

where

c	=	Cohesion property of the soil
N_c	=	Cohesion bearing capacity (see Figure C4-13)
N_q	=	Surcharge bearing capacity factor (see Figure C4-13)
N_γ	=	Density bearing capacity factor (see Figure C4-13)
ζ_c, ζ_q, ζ_γ	=	Footing shape factors (see Table C4-1)
γ	=	Soil density
D	=	Depth of footing
B	=	Width of footing

Table C4-1 Shape Factors for Shallow Foundations (after Vesic, 1975)

Shape of the Base	ζ_c	ζ_q	ζ_γ
Strip	1.00	1.00	1.00
Rectangle	$1 + \frac{B}{L}\frac{N_q}{N_c}$	$1 + \frac{B}{L}\tan\phi$	$1 - 0.4\frac{B}{L}$
Circle and Square	$1 + \frac{N_q}{N_c}$	$1 + \tan\phi$	0.60

For a rehabilitation project, normally some information on footing size and depth might be available; but rarely are the soil properties required for the above calculation readily available. The *Guidelines* allow the calculation of bearing capacity by a qualified geotechnical engineer or the use of conservative presumptive or prescriptive values.

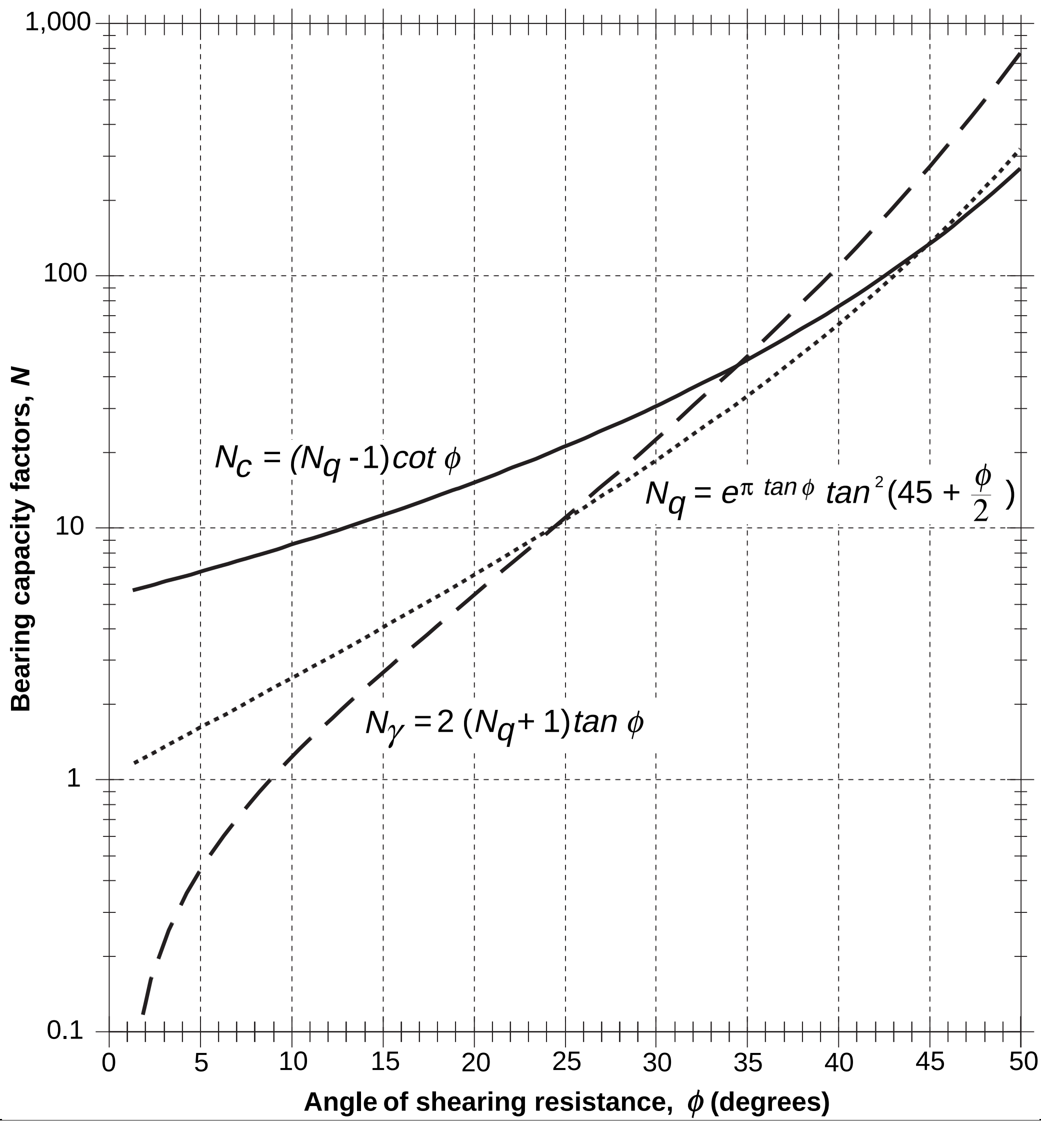


Figure C4-13 Bearing Capacity Factors (calculated from Vesic, 1975)

C4.4.1 Ultimate Bearing Capacities and Load Capacities

Presumptive and prescriptive procedures may be used to determine ultimate load capacities (Q_c) of structures that are located in areas of low seismicity and that are underlain by stable soil conditions (i.e., where a fault rupture, landsliding, and liquefaction are not anticipated). Presumptive ultimate bearing capacities for different foundation soils are provided in Table 4-2. Information developed for Table 4-2 was derived from the *Uniform Building Code* (UBC) and the allowable design values from the UBC were doubled to establish the ultimate bearing pressures for the *Guidelines*. This increase is based upon conventional geotechnical practice, which typically includes a factor of safety of two or more for spread footing foundations.

Alternatively, the ultimate load capacity may be assumed to be equal to 200% or 150% of the dead load, live load, and snow load (that were used for working stress design of the building) acting on a shallow or deep foundation, respectively. The increased uncertainty associated with deep foundations warrants the more conservative factor for these components. Performance of structures during past earthquakes has typically indicated that this empirical rule has provided adequate foundation performance without excessive occurrences of foundation failures, provided that the underlying soils remain stable (i.e., no fault rupture, liquefaction, or landslides).

Site-specific investigation by a qualified geotechnical engineer is the preferred method of determining foundation capacities, particularly for complex analyses.

C4.4.2 Load-Deformation Characteristics for Foundations

C4.4.2.1 Shallow Bearing Foundations

The lateral stiffness and capacity of footings arise from three components, as shown in Figure C4-14. The elastic stiffness solutions shown in Figure 4-2 arise from base contact only, whereas Figure 4-4 provides an elastic stiffness solution generated from passive resistance on the vertical face of the footing. The latter solution (after Wilson, 1988) was derived for bridge abutments, where the soil surface is level with the top of the wall. For buried footings, some judgment is needed in assessing an “equivalent” footing height. For practical purposes, where lateral loads approach the passive pressure, it may be reasonable to assume that the lateral displacement required to mobilize passive pressure is approximately 2% of an “equivalent” footing height (assuming the soil surrounding the footing is dense or stiff). Displacements of approximately 2% to 4% would be more appropriate for softer soils (Clough and Duncan, 1991).

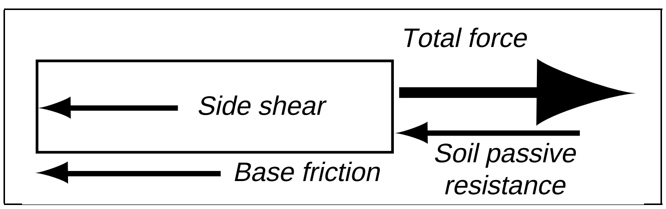


Figure C4-14 Footing Lateral Stiffness and Capacity Components

The determination of displacement as a function of load for a footing is complex (see Figure C4-15). Upon initial loading, the example footing may be relatively stiff as shearing strains are low, or alternatively, until a preconsolidation pressure due to previous overburden or drying (shrinkage) might be reached. At larger deformations, the material may soften progressively until a capacity plateau is reached. If the footing is unloaded, the rebound is usually not complete and permanent displacement occurs. For repeated cyclic loading the permanent displacement can accumulate. When reloaded, the footing can be substantially stiffer than for previous cycles. This information needs to be simplified and generalized for use in a structural analysis model. For this purpose the *Guidelines* promote a strength and stiffness envelope, shown here in Figure C4-15. The lower bound reflects the initial material properties during the first cycle of loading; the upper bound represents the effects of repeated loading. This allows the structural engineer to investigate the sensitivity of the analysis to the soils parameters. It may be that the stiff-strong assumption will give critical results for some structural elements while the flexible-weak will more adversely affect others.

The objective of the force-displacement relationships is to allow the structural engineer to incorporate the foundation characteristics into an analysis model. Consider the spread footing shown in Figure C4-16 with an applied vertical load (P), lateral load (H), and moment (M). The soil characteristics might be modeled as two translational springs and a rotational spring. More common, however, is the use of a Winkler spring model acting in conjunction with foundation structure to eliminate the rotational spring. The conversion to Winkler springs requires the consideration that rotational stiffness may differ substantially from vertical stiffness. Useful discussions of the concepts of rigid and flexible footing behavior are provided by Scott (1981) and Bowles (1982). Note that the values of Winkler or subgrade stiffness coefficients often tabulated in geotechnical textbooks reflect first loading values. Stiffness coefficients for unloading and reloading reflecting cyclic loading conditions can range from about two to five times stiffer, depending on the original density or stiffness of the soil.

A problem frequently encountered in seismic rehabilitation is the analysis of a shear wall or braced frame supported on spread footings. The relationship of the vertical load, overturning moment, and soil properties, and their effect on stiffness and energy dissipation was thoroughly studied by Bartlett (1976). Figure C4-17 illustrates the relationship between overturning moment and base rotation for a wall that is allowed to uplift and/or accommodate compression yielding in the supporting soil medium. This rocking behavior has several important effects on the seismic response of the structure. First of all, rocking results in a decrease in stiffness and lengthening of the fundamental period of the structure. This effect is amplitude-

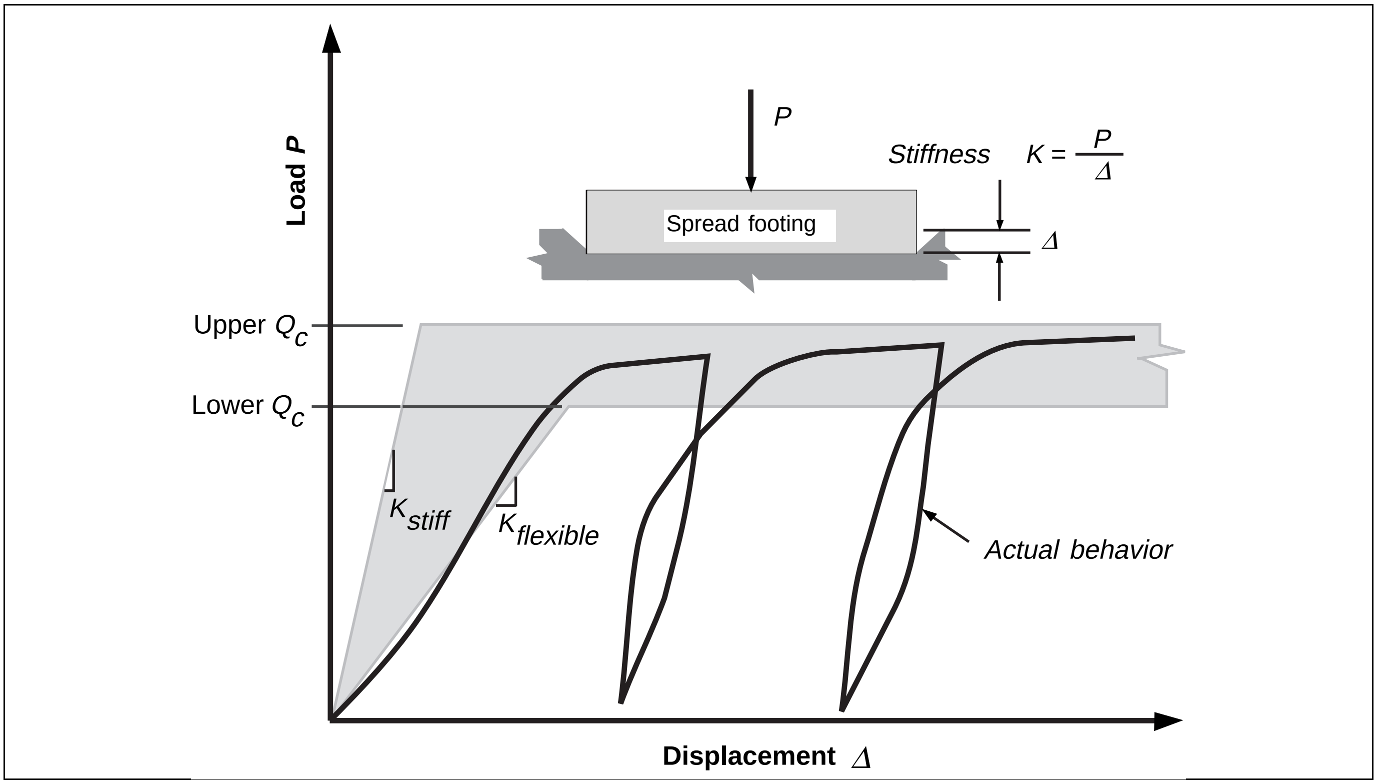


Figure C4-15 Load-Displacement Relationship for Spread Footing

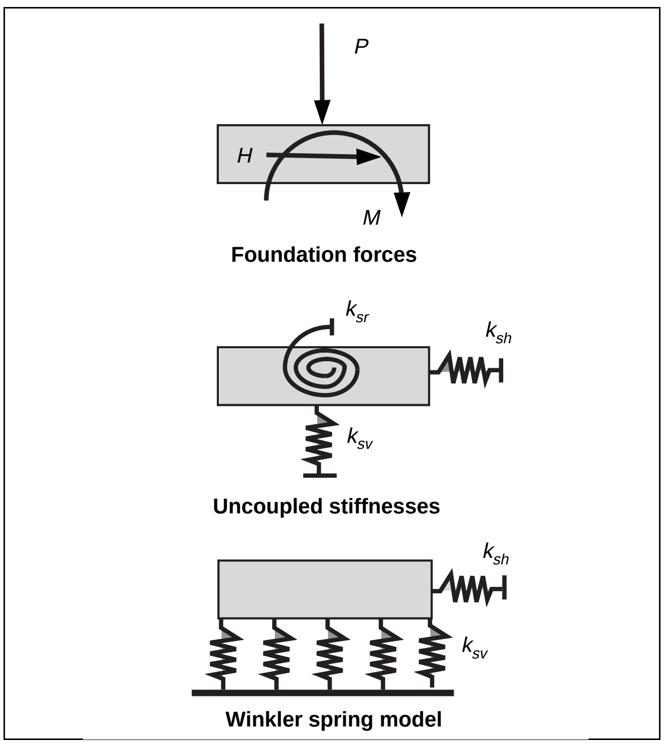


Figure C4-16 Analytical Models for Spread Footing

dependent and therefore highly nonlinear. The result is generally a reduction in the maximum seismic response. Depending on the ratio of initial bearing pressure to the ultimate capacity of the soil, significant amounts of energy may be dissipated by soil yielding. This behavior also can result in increased displacement response of the superstructure and permanent foundation displacements.

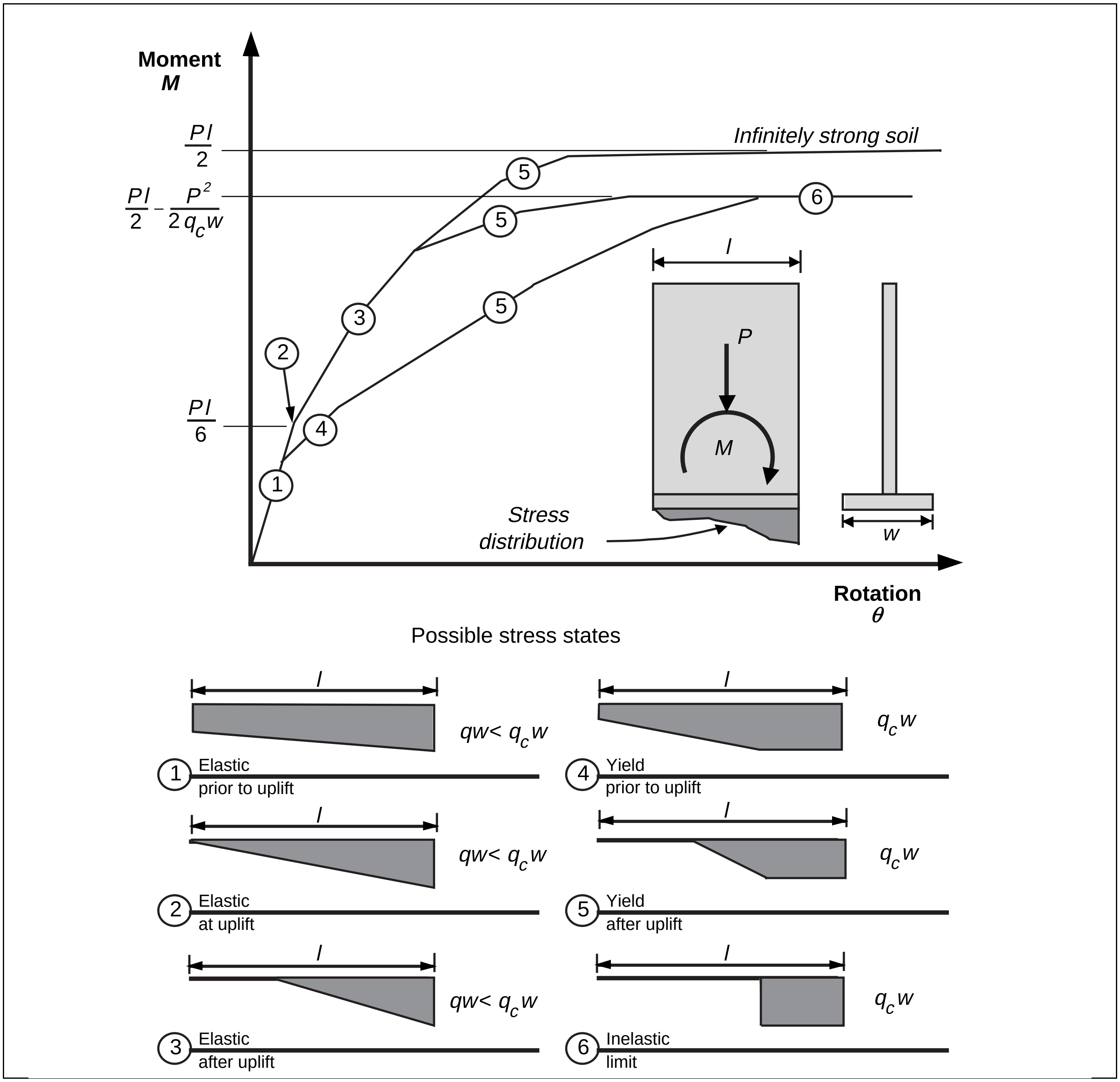


Figure C4-17 Rocking of Shear Wall on Strip Footing

A. Shear Wall and Frame Example

This example illustrates the effects of foundation flexibility on the results of analysis of an eight-story concrete shear wall and frame building, shown in Figure C4-18. The results of an LSP for this structure for both a fixed base and flexible base are summarized below:

Seismicity

Spectral response acceleration at short periods, $S_{XS} = 1.1$

Spectral response acceleration at one second, $S_{X1} = 0.75$

Soil properties

Soil unit weight, $\gamma = 110$ pcf

Shear wave velocity, $v_s = 1100$ ft/sec

Poisson's ratio, $\nu = 0.35$

Initial shear modulus, $G_o = \dfrac{\gamma v_s^2}{g} = 4097$ ksf

Effective shear modulus, $G = 0.35\ G_o = 1434$ ksf

(for $S_{XS}/2.5 = 0.40$ from Table 4-3)

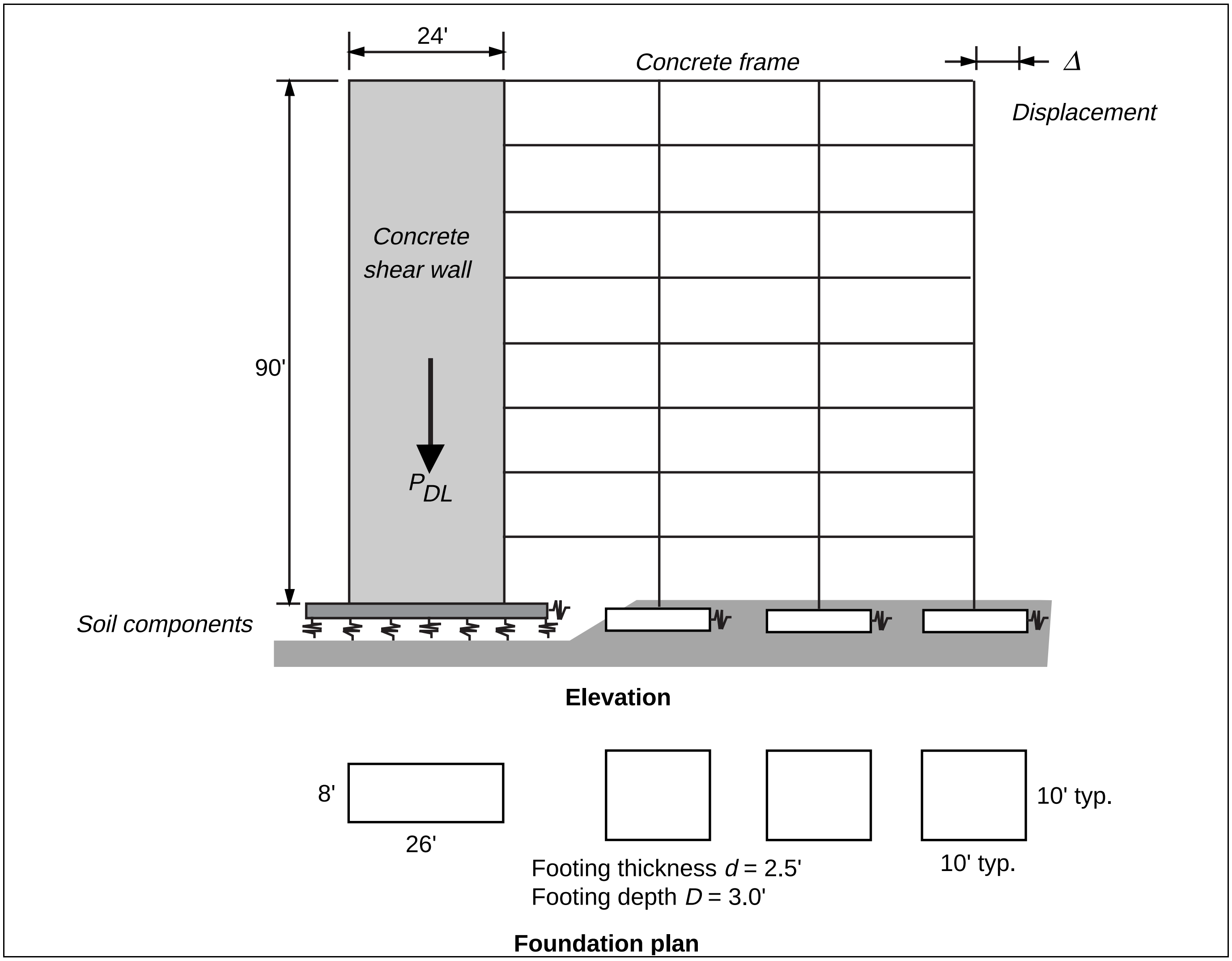


Figure C4-18 Shear Wall and Frame Example

Ultimate bearing capacity, q_c = 12 ksf

Upper bound $q_c = 2(12) = 24$ ksf

Dead load bearing stress available to resist seismic overturning:

$q = 5.85$ ksf for $P_{DL} = 1360$ k and $Q_G = 0.9\, P_{DL}$

Modification factors

$C_1 = C_2 = C_3 = 1.0$

Flexible Foundation Properties

Foundation stiffnesses, in accordance with Gazetas (1991), are:

Lateral stiffness, K_y = 4 footings x K_{yi} = 219,638 k/ft

Rotational stiffness, K_q = shear wall only = 13,155,000 ft-k/rad

Using the SSI procedures from BSSC (1995) (note that the equation for flexible base period presented there contains an error; the equation below is correct):

$$\text{Fixed base stiffness, } k' = 4\pi^2 \frac{W'}{gT^2} = 5229 \text{ k/ft}$$

Flexible base period:

$$T' = T\sqrt{1 + \frac{k'}{K_y}\left[1 + \frac{K_y(0.7h)^2}{K_q}\right]} = 0.93 \text{ sec}$$

	Fixed Base	Flexible Base
Period	0.58 sec	0.93 sec
Base shear	3246 k	2361 k
Overturning moment	194,769 k-ft.	142,368 k-ft.
Roof displacement	19.4 in.	25.9 in.

Checking the fixed base solution, in accordance with the *Guidelines*, Equation 4-11, at the base of the structure reveals that the base overturning moment from the seismic forces unacceptably exceeds twice the plastic capacity of the soil beneath the shear wall.

$$\begin{aligned} Q_C &= M_C = \frac{L}{2} Q_G\left(1 - \frac{q}{q_C}\right) \\ &= \frac{28}{2}(0.9(1360))\left(1 - \frac{5.85}{24}\right) \qquad \text{(C4-3)} \\ &= 12,959 \text{ k-ft} \end{aligned}$$

For a fixed base condition, use force-controlled behavior to determine:

$$\begin{aligned} Q_{UF} &= \frac{Q_E}{C_1 C_2 C_3 J} = \frac{194,769}{(1)(1)(1)(2)} \qquad \text{(C4-4)} \\ &= 97,384 > 12,959 \end{aligned}$$

Although the flexible base overturning moment also greatly exceeds the plastic capacity of the soil, this condition is acceptable, provided that the performance of the structure is acceptable for the increased displacements associated with the rotating foundation beneath the shear wall. Of particular concern in this structure is the ability of the columns of the frame to undergo these displacements without losing vertical-load-carrying capacity. Note that the forces on the structure are reduced significantly by the flexible base assumption, in spite of the larger displacements.

Nonlinear Procedure Results. This example has also been analyzed using the NSP, including the effects of foundation uplift and soil yielding on the inelastic response (Hamburger, 1994). The nonlinear model of the structure included springs representing the stiffness and strength of the soil beneath the shear wall (Figure C4-19). These springs were preloaded with the effect of vertical loads from the structure, but uplift was allowed if the preload was overcome by rotation.

Rocking and compressional soil yielding initiate early in the response of the structure; in fact, it was found that over two-thirds of the deformation demand was absorbed in the foundation soils materials. As a consequence, the inelastic demand on the shear walls was very small, within acceptable limits for the Life Safety Performance Level for the structure as a whole. The stiffness and strength of the soil were varied by factors of 67% and 150% in an effort to test the sensitivity of the analysis results to these parameters. The behavior was not significantly affected, leading to the conclusion that the response is most sensitive to nonlinear rocking itself rather than exact soil properties.

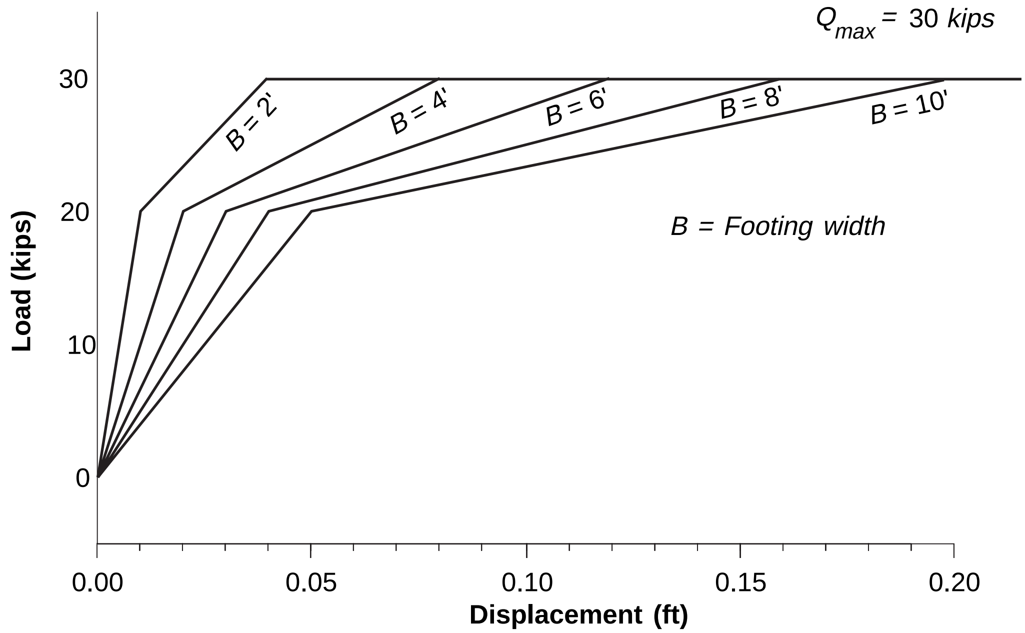


Figure C4-19 Foundation Stiffness and Strength Properties

These nonlinear analysis results have different implications for response than does the linear procedure. The foundation rocking effectively protects the shear walls from large inelastic demand. Modification to the walls and their foundations is not necessary. However, the resulting large lateral movement of the structure could cause undesirable shear failure in some of the columns of the concrete frame. This leads to the conclusion that the columns should be retrofitted to provide greater shear strength, by jacketing or other techniques to provide increased confinement. In contrast, the linear procedure might indicate that a relatively expensive retrofit of the walls and footings is warranted. Perhaps more significantly, the linear procedure with the rigid base assumption might fail to identify the potential problem with the columns.

B. Short Stout Walls on Flexible Grade Beam Example

Figure C4-20 depicts a structural model of one exterior wall of a two-story masonry building (Taner, 1994). The rehabilitation design includes the addition of reinforced concrete shear walls against the unreinforced masonry. A reinforced concrete grade beam couples the three shear wall panels at their base; the tops of the panels are linked together by a bond beam at the roof. The ultimate moment capacity, M_c, of the shear wall panels controls the lateral strength of the structure. Assuming a fixed base for the shear wall panels, displacement at the roof was tolerable at the strength limit state. The designer was concerned, however, that foundation rocking and flexibility might magnify this displacement.

The nonlinear model predicts the incremental displacement, Δ, at the roof due to the interaction of the flexible grade beam with a flexible supporting soil. The model allows unrestrained uplift of the grade beam and footing once the dead load is overcome. The spring constant, k_{sv}, for compressibility of the soil was varied in an effort to assess the sensitivity of the results to this parameter.

The results indicate that significant uplift occurs for any soil stiffness. The distribution and maximum magnitude of foundation contact pressure is highly dependent on the relative stiffness of the soil and the grade beam. The extremely flexible soil virtually allows a rigid body rotation of the structure and a very large incremental roof displacement. The more flexible soils also result in larger moments, M_{max}, in the grade beam. Fortunately, the actual soil is relatively stiff and the incremental displacement is small.

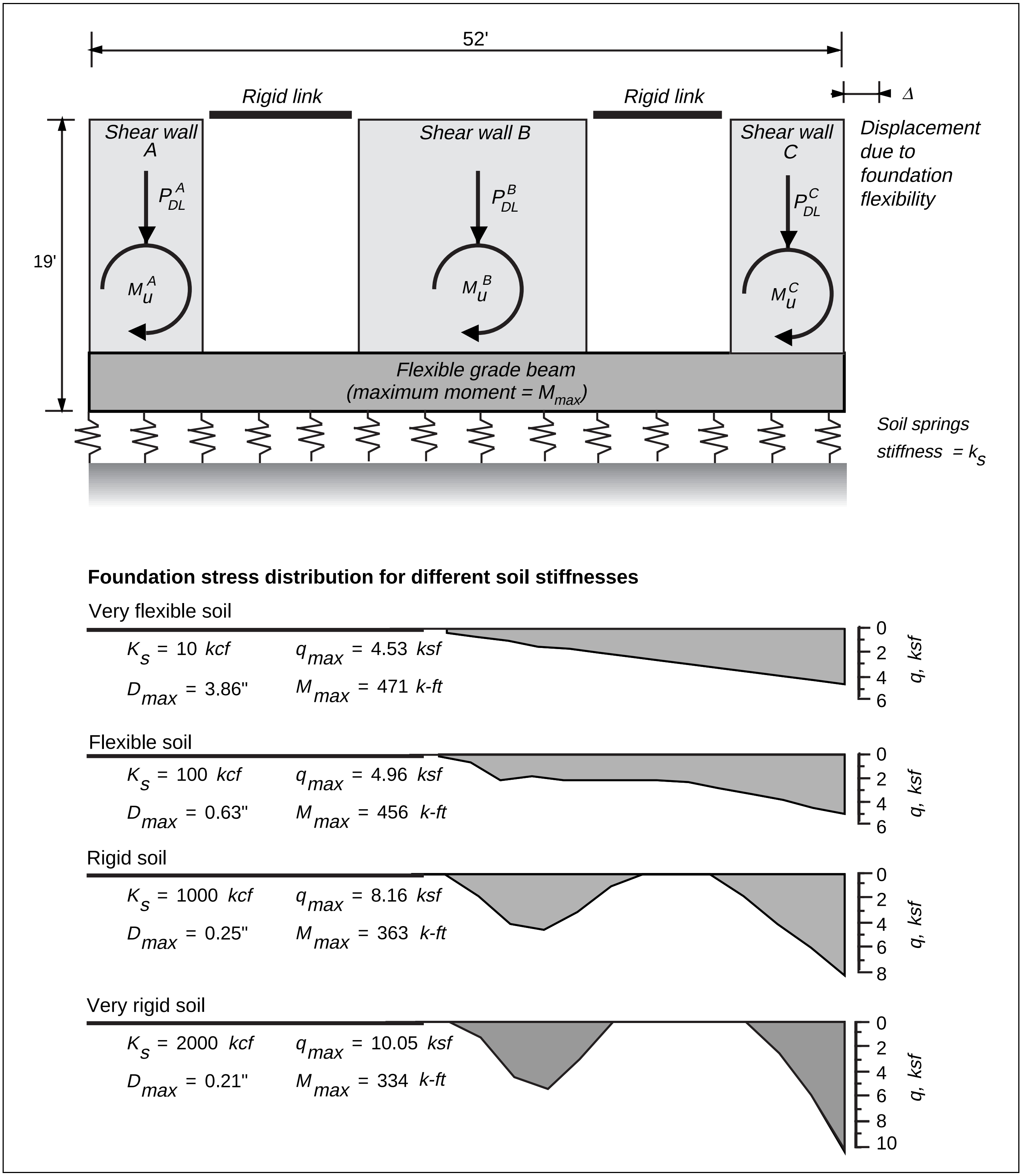


Figure C4-20 Structural Model, One Exterior Wall of Two-Story Masonry Building

C4.4.2.2 Pile Foundations

Axial Loading. Earthquake-induced axial loading of pile groups may be of significant design importance in the analysis of the seismic rocking response of rigid shear walls for buildings when subjected to lateral loading. Analyses also show that the rotational stiffness of a pile group is generally dominated by the axial stiffness of individual piles. The rotational or rocking behavior of a pile group may have a significant influence on the seismic response of a structure and could significantly influence column moments.

Although elastic solutions exist for the pile head stiffness for piles embedded in linear elastic media (Poulos and Davis, 1980; Pender, 1993), the complexities of the nonlinear load transfer mechanisms to the pile shaft and tip make the selection of an equivalent linear elastic modulus for the soil very difficult. The use of the nonlinear Winkler spring approach provides an alternate procedure that has been widely adopted in practice.

The various components of the axial pile load transfer problem are illustrated in Figure C4-21. The overall pile behavior depends on the axial pile stiffness (*AE*) and the load transfer characteristics (*t-z* curves) along the side of the pile and at the pile tip (tip *q-z* curve). The fundamental problem in an analysis of piles under axial loading relates to the uncertainties of the load transfer characteristics at the side and at the pile tip, which in turn influence the pile head load-deflection behavior. Factors that need to be considered in developing the load transfer characteristics include:

- The side-friction capacity along the length of the pile
- The ultimate resistance at the pile tip
- The form of the load transfer-deflection curves associated with each of the above forms of soil resistance

The ultimate capacity of a pile depends on numerous factors, including:

- The soil conditions and pile type
- The geologic history of the site
- The pile installation methods

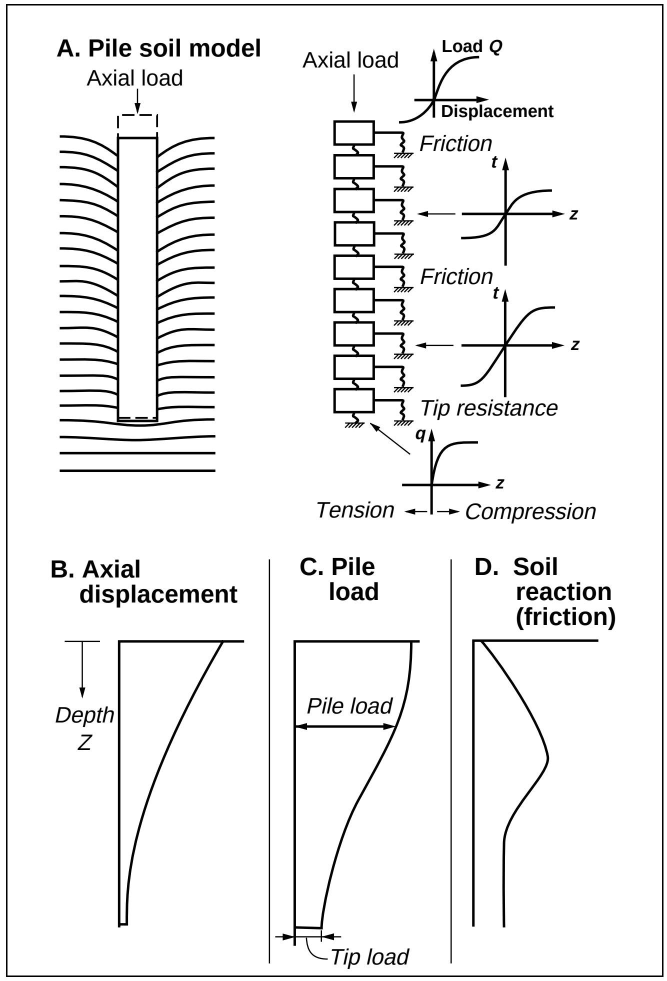


Figure C4-21 ***Schematic Representation of Axial Pile Loading (Matlock and Lam, 1980)***

Numerous methods have been proposed to predict the axial capacity of piles, and can lead to widely varying capacity estimates, as documented in Finno (1989). Incorporation of site-specific pile load test data has been perceived to be the most reliable method for pile capacity determination.

In addition to the ultimate side friction and end-bearing capacity, some assumptions need to be made to develop the load transfer-displacement relationships (for both side friction and end bearing) to evaluate the overall pile behavior. The form of the load transfer-displacement relationship is complex, and there is no uniform agreement on the subject.

A computer approach provides the most convenient means of solving axial pile behavior. Many of the well-established computer programs, such as BMCOL 76 and PILSET (Olsen, 1985), allow for prescription of the *t-z* curves at various depths along the length of the pile (e.g., at the boundaries of each soil layer) and will automatically perform interpolations to develop support curves at all the pile stations. The *t-z* curves for side friction usually are assumed to be symmetrical, and the *q-z* curve at the pile tip usually is assumed to be nonsymmetric.

Uncertainty in axial soil-pile interaction analysis relates largely to uncertainties in soil parameters, including the ultimate pile capacity (skin-friction and end-bearing) and load-displacement relationships. Computers can be used for rigorous nonlinear solutions. However, an approximate nonlinear graphical solution method has been presented by Lam and Martin (1984, 1986). The procedure is shown schematically in Figure C4-22 (for a 70-foot-long, 1-foot-diameter pipe pile embedded in sand, $\phi = 30^{o}$) and involves the following steps:

1. **Soil Load-Displacement Relationships.** Side-friction and end-bearing load-displacement curves are constructed for a given pile capacity scenario (accumulated skin-friction and ultimate tip

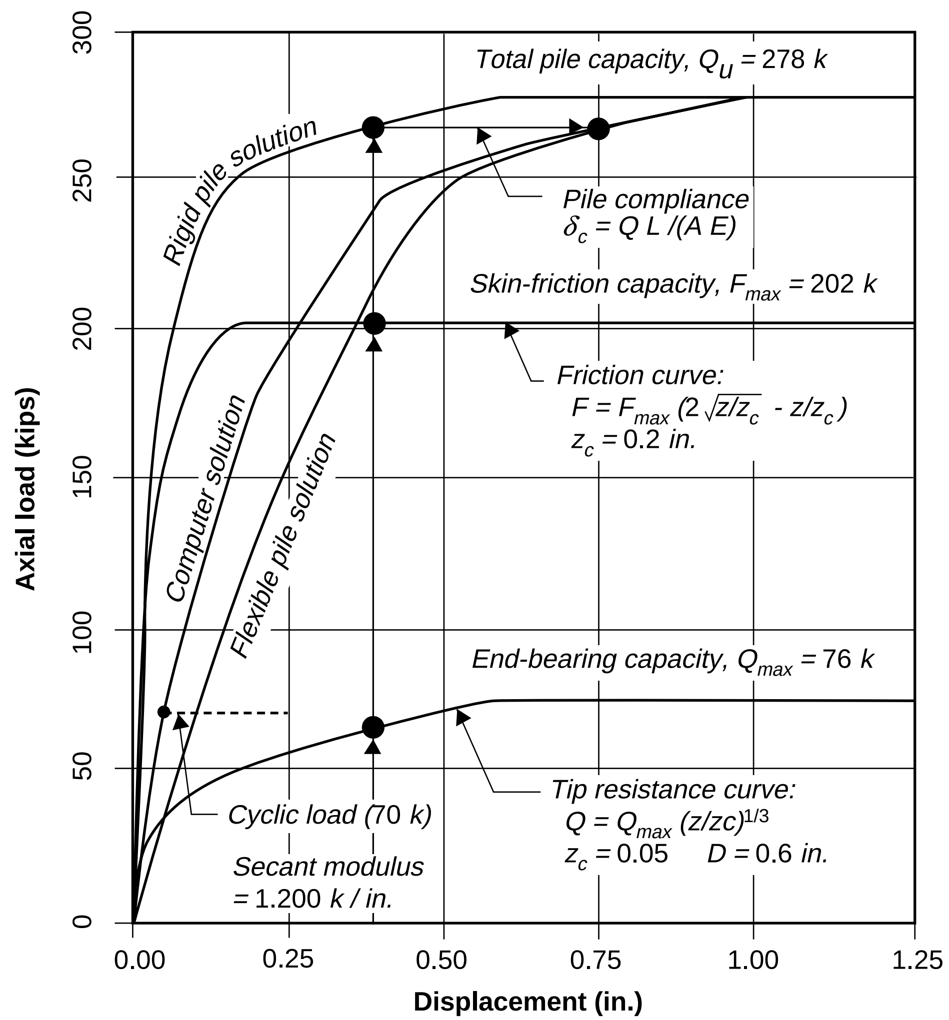


Figure C4-22 Graphical Solution for Axial Pile Stiffness (Lam et al., 1991) resistance). In the example shown, skin friction is assumed mobilized at a displacement of 0.2 inches, and end bearing at a displacement of 0.5 times the pile diameter.

2. **Rigid Pile Solution.** Using the above load-displacement curves, the rigid pile solution can be developed by summation of the side-friction and end-bearing resistance values at each displacement along the load-displacement curves.

3. **Flexible Pile Solution.** From the rigid pile solution, the flexible pile solution can be developed by adding an additional component of displacement at each load level Q to reflect the pile compliance. For the most flexible pile scenario, corresponding to a uniform thrust distribution along the pile shaft, the pile compliance is given by:

$$\delta_c = \frac{QL}{AE} \tag{C4-5}$$

where:

L = Pile length
A = Cross-sectional area
E = Young's modulus of the pile

4. **Intermediate Pile Stiffness Solution.** The "correct" solution, as indicated by the computer solution, is bounded by the rigid pile and flexible pile solutions. In most cases, a good approximation can be developed by averaging the load-displacement curves for the rigid and flexible pile solutions. The above graphical method can be used to solve for the load-displacement curve for any combination of pile/soil situations (end-bearing and friction piles) as well as any pile type or pile material.

As described by Gohl (1993), as an even simpler approximation, pile head stiffness values under normal loading (not exceeding the capacity) may be expressed as some multiple α of AE/L, with the constant α depending on the proportions of shaft and end bearing resistance mobilized. For example, a value of $\alpha = 1.0$ would be appropriate for an end bearing pile on rock with negligible shaft friction. Values of α closer to 2.0 would be reasonable for friction piles with negligible end tip resistance. The range of α from 0.5 to 2.0 in the *Guidelines* encompasses the uncertainties involved with existing foundations, albeit more complex analyses could be used if reliable data are available.

Under earthquake conditions, some magnitude of cyclic axial load will be superimposed on a static bias load (e.g., the static dead weight). Figure C4-23 illustrates the various factors that come into the picture due to a static bias loading. As shown, in a normal design range, where the maximum load level (from superimposing the cyclic load on the static bias) does not exceed the pile capacity (for both the peak compressive or tensile load), the static dead weight can be neglected in solving for the secant stiffness of the pile. The magnitude of cyclic loading, along with the backbone load-displacement curve, can be used to develop the secant stiffness of the pile at the various load levels. However, the load-displacement behavior of the pile will be more complex when the pile capacity (compressive or tensile) is exceeded. Permanent displacement of the pile will occur when the capacity is exceeded.

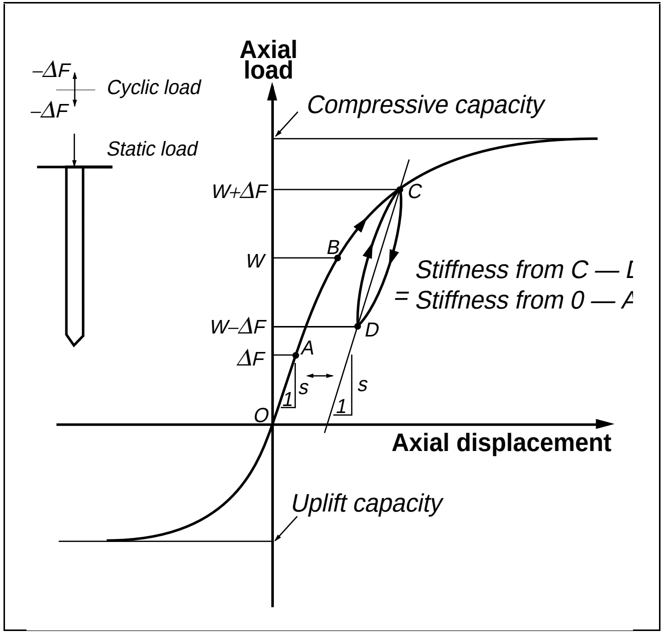


Figure C4-23 ***Load-Displacement Characteristics under Axial Loading (Lam and Martin, 1986)***

Moment-Rotation Capacity. The moment-rotational characteristics and the capacity of a pile footing depend on the following factors:

- The configuration (number of piles and spatial dimension) of the pile footing
- The capacity of each pile for both compression and uplift loading

To illustrate the above concern, Lam (1994) presents an example problem involving a typical pile footing as shown in Figure C4-24. The analyses presented assume a rigid pile cap for the footing, and are quasi-static analyses. The load-displacement curves for each individual pile in the pile group are shown in Figure C4-25. The pile is modeled as an elastic beam-column, and nonlinear axial soil springs are distributed along the pile to represent the soil resistance in both compression and uplift. It can be seen from the figure that the ultimate soil capacities of the pile for

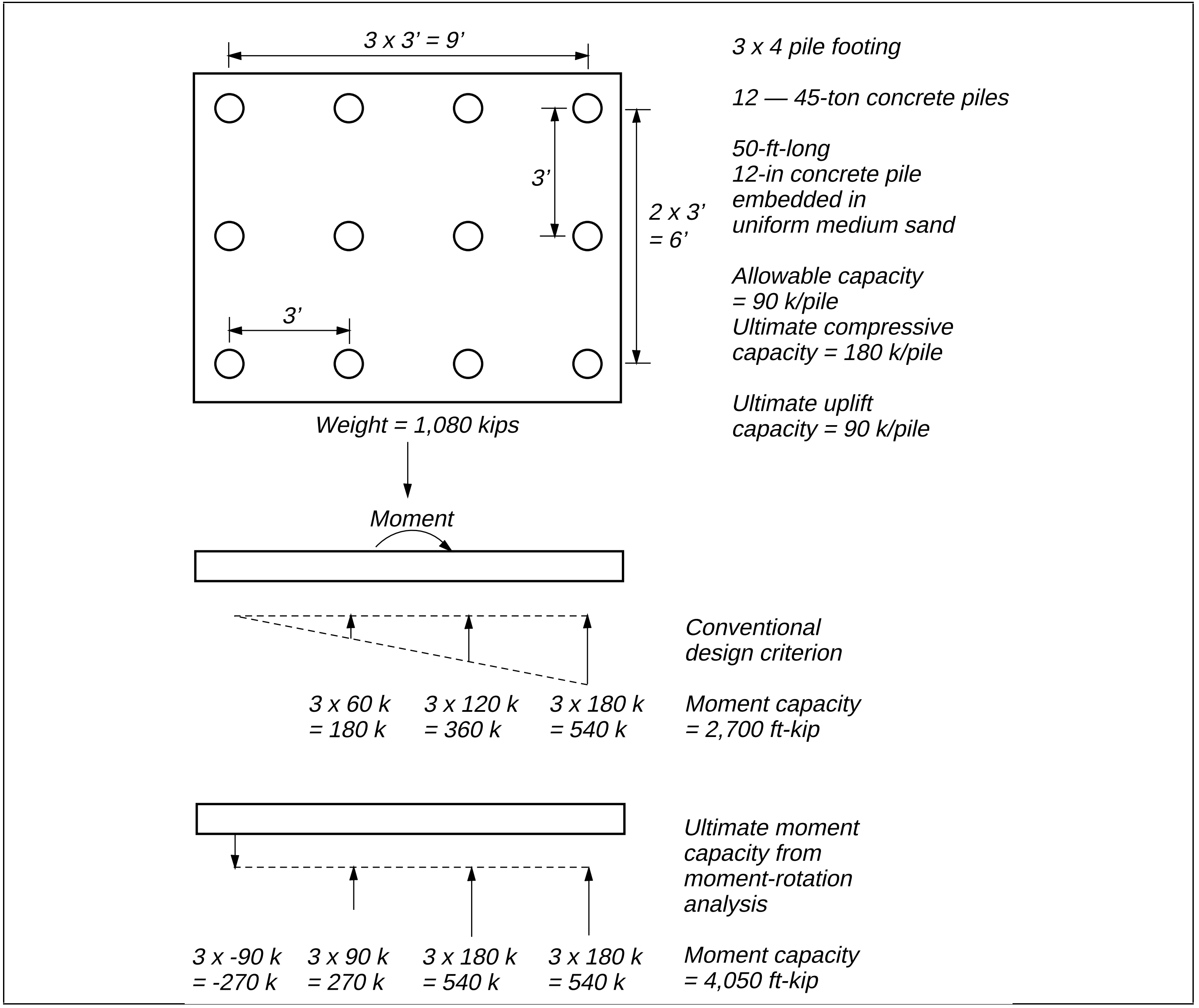


Figure C4-24 Pile Footing Configuration for Moment-Rotation Study (Lam, 1994)

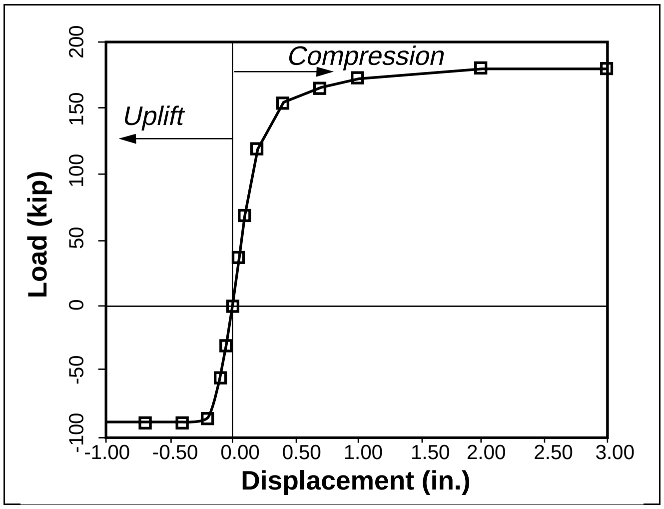


Figure C4-25 Axial Load-Displacement Curve for Single Pile (Lam, 1994)

compression and tension are 180 and 90 kips per pile, respectively, if the connection details and the pile member are adequate to enforce the failure to take place in the soil. The pile has been assumed to be a 50-foot-long, 12-inch concrete pile driven into uniform medium sand, which has a design load capacity of 45 tons per pile. The adopted ultimate capacity values (i.e., 180 kips compression and 90 kips uplift) are the default values commonly assumed by the California Department of Transportation in seismic retrofit projects for the 45-ton class pile. In the example, it is assumed that the footing has been designed for a static factor of safety of 2, or the piles are loaded to half of the ultimate compression capacity prior to the earthquake loading condition.

Figure C4-24 presents various capacity criteria for the pile footing. Under conventional practice, the moment capacity of the pile footing would be 2,700 ft-kip. This capacity arises from assuming a linear distribution in pile reaction across the pile footing. The moment capacity of 2,700 ft-kip is limited by the ultimate compressive capacity value of the most heavily loaded pile (180 kip per pile) while maintaining vertical equilibrium of the overall pile group (i.e., static load of 1,080 kips). The lowest part of Figure C4-24 presents the moment capacity that can be achieved from a nonlinear moment-rotation analysis of the pile footing, in which the moment load increases above the conventional capacity. Nonlinear load-displacement characteristics of the pile are simulated to allow additional load be distributed to the other less loaded

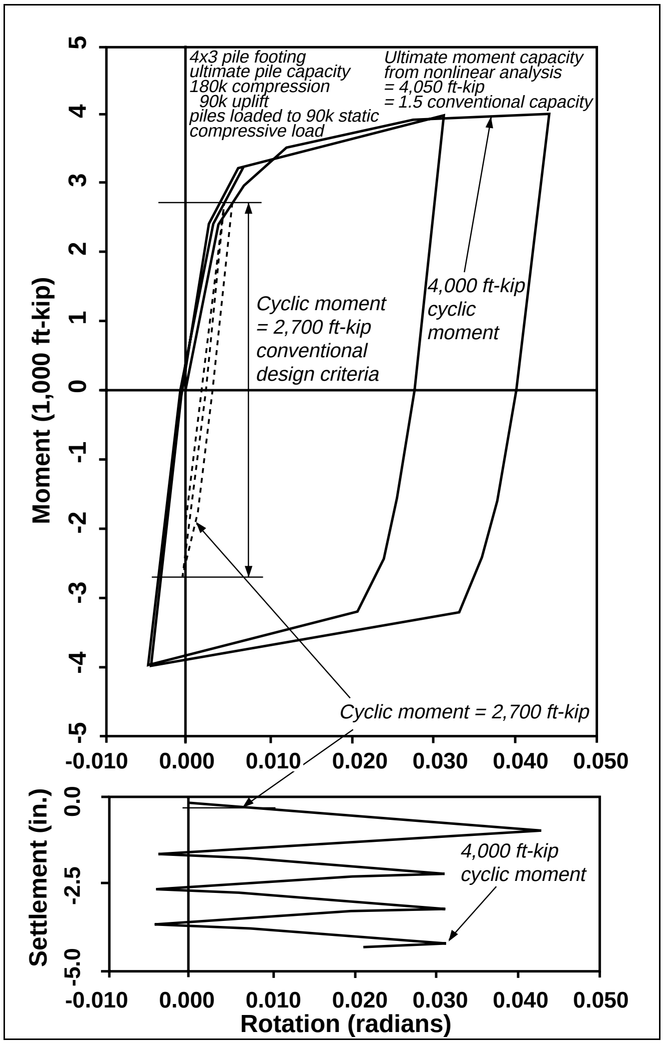


Figure C4-26 Cyclic Moment-Rotation and Settlement-Rotation Solutions (Lam, 1994)

piles in the pile group. As shown, a maximum ultimate capacity of 4,050 ft-kip (1.5 times the conventional capacity) can potentially be achieved by virtue of such nonlinear analysis.

Figure C4-26 presents the cyclic moment-rotation solutions associated with the footing example problem discussed above. The dotted line in the moment-rotation plot defines the monotonic loading path of the moment-rotation relation. Solutions for two uniform cyclic moment loads are presented: a lower cyclic moment level of 2,700 ft-kip corresponding to the conventional design capacity, and a higher cyclic moment load of 4,000 ft-kip. As shown in the figure, at the lower cyclic moment of 2,700 ft-kip, the moment-rotation characteristic is quite linear, and both the moment-rotation characteristics and settlement will equilibrate to the final value very quickly within a few cycles of loading. However, at the higher cyclic moment load of 4,000 ft-kip, progressive settlement of the footing can occur, and within about four cycles of loading, the footing can settle almost five inches. The moment-rotation relationship also indicates that some level of permanent rotation of the footing will likely occur even if the load is symmetric between positive and negative cyclic moments. The potential for the permanent rotation is associated with the change in the state of stress in the soil—from a virgin (unstressed) condition to the equilibrated state—after cyclic loading, unloading, and reloading. A similar analysis, using a static factor of safety of 3 (instead of 2) corresponding to a dead load of 720 kips, resulted in a ultimate moment capacity of 1.3 times the conventional capacity, and a reduced settlement of about 0.25 inches under loading cycles at the increased ultimate capacity level.

Considering the inherent conservatism in pile capacity determinations (especially for compressive loading), most existing pile footings probably have an inherent static factor of safety for dead load of over 3. Hence, it can be speculated that the potential for significant settlement or rotation of a pile footing would not be too high, except for poor soil sites where cyclic degradation of soil strengths can be significant. Typically, the most likely cause of foundation failure would be some form of permanent rotation of the pile group if the size of the footing and the number of piles are inadequate. Therefore, it is important to have a better appreciation of the magnitude of foundation rotation that is tolerable by the pile-supported structure, particularly for retrofit seismic design—where unnecessary conservation can be expensive.

A state-of-the-practice commentary on the seismic design of pile foundations, including a discussion of design uncertainties and structural design issues, has been presented by Martin and Lam (1995). A useful computer program, suitable for determining lateral, moment, and axial stiffness parameters for a vertical pile group, has been documented by Reese and others (1994). For battered pile systems, the computer program PILECAP has been developed for assembling a pile cap stiffness matrix, and is documented by Lam and Martin (1986).

C4.4.2.3 Drilled Shafts

No commentary is provided for this section.

C4.4.3 Foundation Acceptability Criteria

Geotechnical parts and actions of foundations are those whose behavior is characterized by the properties of the soil materials supporting the building. Bearing pressures beneath spread footings or friction forces on a pile are examples of geotechnical actions. These are differentiated from structural actions—such as the bending of a concrete footing, or the compression capacity of a steel pile—covered in other chapters. As with other elements and components, the acceptability of geotechnical parts depends on the performance goal for the building. Additionally, however, the basic procedure for rehabilitation, and the specific assumptions used in the analysis of the building, limit the use of the results with respect to foundation parts.

C4.4.3.1 Simplified Rehabilitation

Chapter 10 presents Simplified Rehabilitation appropriate for use on some buildings. These procedures include some investigation of foundation conditions and, in some cases, requirements for basic modifications.

C4.4.3.2 Linear Procedures

If the foundation is assumed to be fixed in the analysis, geotechnical component displacements are, by definition, zero. Thus, for these actions, acceptability can only be assessed by considering the geotechnical components to be force-controlled. This reduces the seismic force contribution to a more realistic level. Since geotechnical components are actually “ductile” in contrast to most other force-controlled components, acceptable force levels for these fixed-base actions may be based on upper-bound capacities. If these capacities are exceeded, the implication is that actual geotechnical component displacements may be large enough to increase displacement demands significantly in other parts of the structure. The practical consequence is to require the designer to model the elastic properties of the foundation.

If the analysis includes elastic modeling of the foundation, then for shallow and deep foundations, no limit of uplift or compression displacement is necessary for Collapse Prevention or Life Safety Performance Levels. In essence, *m* = infinity for these cases. This is reasonable, since soil bearing capacity does not degrade for short-term cyclic loads and the consequences of foundation movements are reflected in an approximate manner by the response of the structure in the model. This is true even though fictitious “tension” is allowed to develop between a footing and the soil. This is considered to be analogous to tension yielding in bending of a structural element where the estimate of inelastic displacements assumes that the beam remains elastic. Even if the seismic overturning moment is equal to the maximum resisting moment due to gravity, this situation changes quickly with seismic load reversal. Experience with past earthquakes does not indicate that gross overturning is a problem for buildings. If the calculated displacements do not result in adverse behavior in the structure, there is no need to limit foundation displacements.

However, the situation for the Immediate Occupancy Performance Level is different, since foundation displacements may result in damage that impedes the use of the facility. For this reason, fixed-base conditions should not be assumed for structures sensitive to base movement.

C4.4.3.3 Nonlinear Procedures

The assumption that the base of the structure is rigid in nonlinear procedures is acceptable, provided that the resulting forces do not exceed upper-bound component capacities. The rationale for this limitation is similar to that for linear procedures.

If the foundation is modeled with appropriate nonlinear force-displacement relationships, the acceptability of geotechnical components for Collapse Prevention or Life Safety Performance is analogous to that for linear procedures. For Immediate Occupancy, the amount of the total structural displacement due to foundation movement must be calculated. Some percentage of this foundation-related movement is assumed to be permanent, and the effects of this must be included in considering whether the building can remain functional. Permanent foundation movement is controlled by foundation soil type and thickness, and foundation system characteristics (footing dimensions and geometry).

C4.5 Retaining Walls

The equation in the *Guidelines* for the seismic increment of earth pressure acting on a building retaining wall is a rounded-off form of the equation developed by Seed and Whitman (1970). (In their equation, the fraction 3/8 rather than the rounded-off decimal 0.4 is used. In view of the uncertainty in these pressures, the rounding off is justified.) This equation was developed as an approximation of a seismic earth pressure formulation presented by Seed and Whitman ("Mononobe-Okabe method," 1970) for yielding (free-standing) retaining walls. Because building walls retaining soil (e.g., basement walls) are relatively nonyielding due to the restraint provided by the interior floors, the applicability of these equations to building walls is a matter of some debate. Alternative elastic solutions for seismic wall pressures have been proposed. The most widely used elastic solution is that of Wood (1973), which provides seismic pressures of the order of twice those given by the Seed and Whitman expression. The argument for the lower values of the Seed and Whitman expression is that a limited number of dynamic finite element analyses and one case history (Chang et al., 1990) have found that the calculated and observed seismic earth pressures were of the same order of magnitude as those given by the Mononobe-Okabe formulations and lower than those of the Wood elastic solutions. In a state-of-the-art paper, Whitman (1991) concluded that the Mononobe-Okabe equation should suffice for nonyielding walls, except for the case where a structure, founded on rock, has walls retaining soil. Other publications that discuss seismic lateral earth pressures include Martin (1993), Soydemir (1991), and the *ASCE Standard 4* (ASCE, 1986; under revision).

If building retaining walls are required to be utilized as part of the foundation system to resist seismically-induced structure inertia forces, then higher pressures may be required to be developed on the walls. The maximum pressures that can be mobilized by the soil are passive earth pressures. Because of uncertainty regarding the direction or significance of soil inertia forces affecting the passive pressure capacity, it is suggested that passive pressures be obtained using conventional static earth pressure formulations.

C4.6 Soil Foundation Rehabilitation

Foundation enhancements may be required because of inadequate capacity of existing foundations to resist overturning effects (inadequate footing bearing capacities) or inadequate shear resistance of the foundations. Additionally, foundation enhancements may be required to support structural improvements, such as new shear walls or strengthening of existing shear walls. In either event, the foundation enhancements may be accomplished by a combination of one or several of the following schemes:

- Soil improvement

- Footing improvement (new footing/enlargement of existing footing)
- Foundation underpinning

C4.6.1 Soil Material Improvements

Foundation soil improvements may be undertaken to address global concerns, such as the development of liquefaction, or to improve bearing capacity of the underlying foundation soils. Compaction grouting or chemical grouting are likely choices in either scenario. The level of foundation improvement with either technique may require field testing to verify that the density of soil has improved to the desired level and the extent of grout permeation is consistent with design objectives. Because of the difficulty of working beneath the existing structures to accomplish this goal of soil improvement, a test program may be needed to first verify the procedure and then establish realistic criteria for the level of soil improvement. This may need to be done well in advance of design to indicate the feasibility and economics of these solutions.

C4.6.2 Spread Footings and Mats

Footing improvements could include both constructing new footings to support new shear walls or columns for the structural retrofitting, and enlarging existing footings to support improvements to existing shear walls or additional loads anticipated through the existing shear walls. In either event, planners of the new construction will need to evaluate the relative impact of the new addition (new footing or enlarged footing) upon the existing structure to determine whether the new construction will induce settlements that may affect the integrity of the existing structure.

Footing underpinning is another solution that may be used to resist overturning effects. This solution may typically involve construction of micropiles around the perimeter of an existing footing, then the casting of a grade beam/pile cap integrally with the existing footing. Micropiles may range in size from three inches to as much as eight inches in diameter. Load capacities of the micropiles will vary depending upon subsurface soil conditions; however, load capacities on the order of 50 to 100 tons are not uncommon. This type of foundation strengthening may be used to resist both compression and tension loads, provided that the micropiles are adequately designed and installed in an appropriate bearing stratum. However, the evaluation of this foundation enhancement must consider that the stiffness of the micropiles is much greater than that of the spread footing foundation; the micropiles will deflect less—and thereby attract more—foundation loads than did the original spread footing foundation. This difference in stiffness must be considered in the structural analysis.

C4.6.3 Piers and Piles

No commentary is provided for this section.

C4.7 Definitions

No commentary is provided for this section.

C4.8 Symbols

No commentary is provided for this section.

C4.9 References

ASCE, 1986 (revision in press, 1995), “Seismic Analysis of Safety-Related Nuclear Structures and Commentary,” *ASCE Standard 4*, American Society of Civil Engineers, New York, New York.

Bartlett, P. E., 1976, *Foundation Rocking on a Clay Soil*, Report No. 154, School of Engineering, University of Auckland, New Zealand.

Bartlett, S. F., and Youd, T. L., 1992, “Empirical Prediction of Lateral Spread Displacement,” edited by Hamada, M., and O'Rourke, T. D., *Proceedings of the Fourth Japan-U.S. Workshop on Earthquake Resistant Design of Lifeline Facilities and Countermeasures for Soil Liquefaction*, Report No. NCEER-92-0019, National Center for Earthquake Engineering Research, Buffalo, New York, Vol. I, pp. 351–365.

Baziar, M. H., Dobry, R., and Elgamal, A-W.M., 1992, *Engineering Evaluation of Permanent Ground Deformations Due to Seismically-Induced Liquefaction*, Report No. NCEER-92-0007, National Center for Earthquake Engineering Research, Buffalo, New York.

Bell, J. W., 1984, *Quaternary Fault Map of Nevada-Reno Sheet*, Nevada Bureau of Mines and Geology Map 79.

Bowles, J. E., 1982, *Foundation Analysis and Design*, McGraw-Hill, New York, New York.

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.

CDMG, 1975, *Guidelines for Evaluating the Hazard of Surface Fault Rupture*, California Division of Mines and Geology Note 49, Sacramento, California.

Chang, C. Y., Power, M. S., Mok, C. M., Tang, T. K., and Tang, H. T., 1990, "Analysis of Dynamic Lateral Earth Pressures Recorded on Lotung Reactor Containment Model Structure," *Proceedings from the Fourth U.S. National Conference on Earthquake Engineering*, Vol. 3, Palm Springs, California, Earthquake Engineering Research Institute, Oakland, California.

Clough, G. W., and Duncan, J. M., 1991, "Earth Pressures," Foundation Engineering Handbook, edited by Fang, H. Y., Van Nostrand Reinhold, New York, New York.

Cofer, W. F., McLean, E. I., and McGuire, J. W., 1994, *Analytical Modeling of Foundations for Seismic Analysis of Bridges*, Report NO. WA-RD 328.2, Washington State Transportation Center, Washington State University, Pullman, Washington.

Comartin, C. D., Keaton, J. R., Grant, W. P., Martin, G. R., and Power, M. S., 1994, "Transitions in Seismic Analysis and Design Procedures for Buildings and Their Foundations," *Proceedings of the Sixth U.S.-Japan Workshop on the Improvement of Structural Design and Construction Practices*, Report No. ATC 15-5, Applied Technology Council, Redwood City, California.

Evans, M. D., 1987, *Undrained Cyclic Triaxial Testing of Gravels - The Effect of Membrane Compliance*, Ph.D. thesis, Department of Civil Engineering, University of California, Berkeley, California.

Fear, C. E., and McRoberts, E. C., 1995, "Reconsideration of Initiation of Liquefaction in Sandy Soils," *Journal of the Geotechnical Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 121, No. GT3, pp. 249–260.

Finno, R. J., editor, 1989, *Predicted and Observed Axial Behavior of Files*, Geotechnical Special Publication No. 23, American Society of Civil Engineers, New York, New York.

Franklin, A. G., and Chang, F. K., 1977, *Earthquake Resistance of Earth and Rock-Fill Dams: Permanent Displacements of Earth Embankments by Newmark Sliding Block Analysis*, Miscellaneous Paper S-71-17, Report 5, U.S. Army Corps of Engineers, Waterways Experiment Station, Vicksburg, Mississippi.

Gazetas, G., 1991, "Foundation Vibrations," *Foundation Engineering Handbook*, edited by Fang, H.-Y., Van Nostrand Reinhold, New York, New York, pp. 553–593.

Girault, P. D., 1986, "Analyses of Foundation Failures," *Proceedings of International Conference on the Mexico Earthquakes - 1985 - Factors Involved and Lessons Learned*, American Society of Civil Engineers, New York, New York.

Gohl, W. B., 1993, *Response of Pile Foundations to Earthquake Shaking—General Aspects of Behavior and Design Methodologies*, Seismic Soil/Structure Interaction Seminar, Canadian Society for Civil Engineering/Vancouver Structural Engineers Group, Vancouver, British Columbia.

Hamada, M., Yasuda, S., Isoyama, R., and Emoto, K., 1986, *Study on Liquefaction Induced Permanent Ground Displacements*, Report for the Association for the Development of Earthquake Prediction, Tokyo, Japan.

Hamburger, R. O., 1994, *Investigation of Foundation Rocking Effects, Escondido Village Midrises*, ATC 33 Project Working Paper (unpublished), Applied Technology Council, Redwood City, California.

Hanna, A. M., 1981, "Foundations on Strong Sand Overlying Weak Sand," *Journal of the Geotechnical Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 107, No. GT7, pp. 915–927.

Hanna, A. M., and Meyerhof, G. G., 1980, "Design Charts for Ultimate Bearing Capacity of Foundations on

Sand Overlying Soft Clay," *Canadian Geotechnical Journal*, Vol. 17, pp. 300–303.

Hart, E. W., Bryant, W. A., Smith, T. C., Bedrossian, T. L., and Smith, D. P., 1981, *Summary Report: Fault Evaluation Program 1979–1980 (Southern San Francisco Bay Region)*, California Division of Mines and Geology Open-File Report 81-3SF, Sacramento, California.

Hecker, S., 1993, "Quaternary Tectonics of Utah with Emphasis on Earthquake-Hazard Characterization," *Utah Geological Survey Bulletin* 127.

Housner, G. W., 1963, "The Behavior of Inverted Pendulum Structures," *Bulletin of the Seismological Society of America*, Vol. 53, No. 2, pp. 403–417.

Hucklebridge, A. A., and Clough, R. W., 1978, "Seismic Response of Uplifting Building Frames," *Journal of the Structural Division*, American Society of Civil Engineers, Vol. 104, No. ST8, pp. 1211–1229.

Hynes, M. E., 1988, *Pore Pressure Generation Characteristics of Gravel under Undrained Cyclic Loading*, Ph.D. thesis, Department of Civil Engineering, University of California, Berkeley, California.

Ishihara, K., and Yoshimine, M., 1992, "Evaluation of Settlements in Sand Deposits Following Liquefaction During Earthquakes," *Soils and Foundations*, Japanese Society of Soil Mechanics and Foundation Engineering, Vol. 32, No. 1, pp. 173–188.

Jennings, P. W., 1992, *Preliminary Fault Activity Map of California*, DMG Open-File Report 92-03, California Division of Mines and Geology, Sacramento, California.

Kariotis, J. C., El-Mustapha, A. M., and Ewing, R. D., *Influence of Foundation Model on the Uplifting Response of Structures*, Report No. 2.3-2, U.S.-Japan Coordinated Program for Masonry Building Research, Ewing/Kariotis/Englekirk & Hart, Los Angeles, California.

Lam, I. P., 1994, "Soil-Structure Interaction Related to Piles and Footings," *Proceedings, Second International Workshop on the Seismic Design of Bridges*, Queenstown, New Zealand.

Lam, I. P., and Martin, G. R., 1984, "Seismic Design for Highway Bridge Foundations," *Proceedings*, *Lifeline Earthquake Engineering Performance, Design and Construction*, American Society of Civil Engineers, San Francisco, California.

Lam, I. P., and Martin, G. R., 1986, *Seismic Design of Highway Bridge Foundations*, Reports FMWA/RD-86/101, 102, and 103, Federal Highway Administration, Washington, D.C.

Lam, I. P., Martin, G. R., and Imbsen, R., 1991, "Modeling Bridge Foundations for Seismic Design and Retrofitting," *Transportation Research Record*, Washington, D.C., No. 1290.

Ledbetter, R. H., 1985, *Improvement of Liquefiable Foundation Conditions Beneath Existing Structures*, Technical Report REMR-GT-2, U.S. Army Corps of Engineers, Waterways Experiment Station, Vicksburg, Mississippi.

Lee, K. L., and Albaisa, A., 1974, "Earthquake Induced Settlement in Saturated Sands," *Journal of the Soil Mechanics and Foundation Division*, American Society of Civil Engineers, New York, New York, Vol. 100, No. GT4, pp. 387–406.

Levine, M. B., and Scott, R. F., 1989, "Dynamic Response Verification of Simplified Bridge-Foundation Model," *Journal of the Geotechnical Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 115, No. 2.

Makdisi, F. I., and Seed, H. B., 1978, "Simplified Procedure for Estimating Dam and Embankment Earthquake-Induced Deformations," *Journal of the Geotechnical Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 104, No. GT7, pp. 849–867.

Marcuson, W. F., and Hynes, M. E., 1990, "Stability of Slopes and Embankments During Earthquakes," *Proceedings of ASCE/Pennsylvania*, DOT Geotechnical Seminar, Hershey, Pennsylvania.

Martin, G. R., 1993, "Seismic Design of Retaining Structures," *Proceedings, Seismic Soil Structure Interaction Seminar*, Vancouver, British Columbia.

Martin, G. R., and Lam, I. P., 1995, "Seismic Design of Pile Foundations: Structural and Geotechnical Issues," *Proceedings, Third International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics*, St. Louis, Missouri, Vol. III, pp. 1491–1515.

Martin, G. R., and Yan, L., 1995, "Modeling Passive Earth Pressure for Bridge Abutments," presented at ASCE Conference, San Diego, California.

Meek, J. W., 1978, "Effects of Foundation Tipping in Dynamic Response," *Journal of the Structural Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 6, No. 5, pp. 437–454.

Mendoza, M. J., and Auvinet, G., 1988, "The Mexico Earthquake of September 19, 1985–Behavior of Building Foundations in Mexico City," *Earthquake Spectra*, Earthquake Engineering Research Institute, Oakland, California, Vol. 4, No. 4.

Meyerhof, G. G., 1974, "Ultimate Bearing Capacity of Footings on Sand Layer Overlying Clay," *Canadian Geotechnical Journal*, Vol. 11, No. 2, pp. 223–229.

Mitchell, J. K., 1981, "Soil Improvement: State-of-the-Art," *State of the Art Report, Session 12, Tenth International Conference on Soil Mechanics and Foundation Engineering*, Stockholm, Sweden.

Mitchell, J. K., 1991, "Site Remediation Methodologies," *Seismic Short Course on Evaluation and Mitigation of Earthquake-Induced Liquefaction Hazards*, San Francisco State University (unpublished), San Francisco, California.

Mitchell, J. K., Masood, T., Kayen, R. E., and Seed, R. B., 1990, *Soil Conditions and Earthquake Hazard Mitigation in the Marina District of San Francisco*, Report No. UCB/EERC - 90/08, Earthquake Engineering Research Center, University of California, Berkeley, California.

Mitchell, J. K., and Tseng, D.-J., 1990, "Assessment of Liquefaction Potential by Cone Penetration Resistance," *Proceedings, H. Bolton Seed Memorial Symposium*, BiTech Publishers, Vancouver, British Columbia, Vol. 2, pp. 335–350.

Nakaki, D. K., and Hart, G. C., 1987, *Uplifting Response of Structures Subjected to Earthquake Motions*, Report No. 2.1-3, U.S.-Japan Coordinated Program for Masonry Building Research, Ewing/Kariotis/Englekirk & Hart, Los Angeles, California.

Nakata, J. K., Wentworth, C. M., and Machette, M. N., 1982, *Quaternary Fault Map of the Basin and Range and Rio Grande Rift Provinces, Western United States*, Open-File Report 82-579, U.S. Geological Survey.

National Research Council, 1985, *Liquefaction of Soils During Earthquakes*, Committee on Earthquake Engineering, Commission on Engineering and Technical Systems, National Academy Press, Washington, D.C.

Newmark, N. M., 1965, "Effect of Earthquake on Dams and Embankments," *Geotechnique*, London, Vol. 15, No. 2, pp. 139–160.

Olsen, R. E., 1985, *PILSET Program User's Manual*, Department of Civil Engineering, University of Texas at Austin, Texas.

Pender, M. J., 1993, "A Seismic Pile Foundation Design Analysis," *Bulletin of the New Zealand National Society for Earthquake Engineering*, Vol. 26, No. 1, pp. 49–161.

Personius, S. F., and Scott, W. E., 1990, *Preliminary Surficial Geologic Map of the Salt Lake City Segment and Parts of Adjacent Segments of the Wasatch Fault Zone, Davis, Salt Lake, and Utah Counties, Utah*, Map MF-2114, U.S. Geological Survey.

Pfeiffer, T. J., and Higgins, J. D., 1991, *Colorado Rockfall Simulation Program User's Manual*, Publication CDOT-DTD-ED3/CSM-89-2B, Colorado Department of Transportation, Denver, Colorado.

Poulos, H. G., and Davis, E. H., 1980, *Pile Foundation Analysis and Design*, John Wiley & Sons, New York, New York.

Psycharis, I. N., 1982, *Dynamic Behavior of Rocking Structures Allowed to Uplift*, Report No. EERL-81-02, Earthquake Engineering Research Laboratory, California Institute of Technology, Pasadena, California.

Pyke, R., Seed, H. B., and Chan, C. K., 1975, "Settlement of Sands under Multidirectional Shaking," *Journal of the Geotechnical Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 101, No. 4.

Reese, L. C., Wang, S. T., Awashika, K., and Lam, P.H.F., 1994, *GROUP—A Computer Program for Analysis of a Group of Piles Subjected to Axial and Lateral Loading*, Ensoft, Inc., Austin, Texas.

Robertson, P. K., and Campanella, R. G., 1985, “Liquefaction of Sands Using the CPT,” *Journal of the Geotechnical Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 111, No. GT3, pp. 384–403.

Robertson, P. K., Woeller, D. J., and Finn, W.D.L., 1992, “Seismic Cone Penetration Test for Evaluating Liquefaction Potential under Cyclic Loading,” *Canadian Geotechnical Journal*, Vol. 29, No. 4, pp. 686–695.

Roesset, J. M., Whitman, R. V., and Droby, R., 1973, “Modal Analysis for Structures with Soil Structure Interaction,” *Journal of the Structural Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 99, No. ST3, pp. 399–416.

Rutenberg, A., Jennings, P. C., and Housner, G. W., 1982, “The Response of Veterans Hospital Building 41 in the San Fernando Earthquake,” *Earthquake Engineering and Structural Dynamics*, Vol. 10, pp. 359–379.

Scott, R. F., 1981, *Foundation Analysis*, Prentice-Hall, Englewood Cliffs, New Jersey.

Seed, H. B., and De Alba, P., 1986, “Use of SPT and CPT Tests for Evaluating the Liquefaction Resistance of Sands,” *Proceedings*, *INSITU '86*, *ASCE Special Conference on Use of In Situ Testing in Geotechnical Engineering*, Geotechnical Special Publication No. 6, Virginia Polytechnical Institute and State University, Blacksburg, Virginia, pp. 281–302.

Seed, H. B., and Idriss, I. M., 1971, “Simplified Procedure for Evaluating Soil Liquefaction Potential,” *Journal of the Soil Mechanics and Foundations Division*, American Society of Civil Engineers, New York, New York, Vol. 97, No. SM9, pp. 1249–1273.

Seed, H. B., and Idriss, I. M., 1982, “Ground Motions and Soil Liquefaction During Earthquakes,” *Monograph Series*, Earthquake Engineering Research Institute, Oakland, California.

Seed, H. B., and Whitman, R. V., 1970, “Design of Earth Retaining Structures for Dynamic Loads,” *ASCE Specialty Conference - Lateral Stresses in the Ground and Design of Earth Retaining Structures*, New York, New York, pp. 103–147.

Seed, H. B., Tokimatsu, K., Harder, L. F., and Chung, R. M., 1985, “Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations,” *Journal of the Geotechnical Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 111, No. 12, pp. 1425–1445.

Seed, R. B., and Harder, L. F., 1990, “SPT-based Analysis of Cyclic Pore Pressure Generation and Undrained Residual Strength,” *Proceedings of the H. B. Seed Memorial Symposium*, Vol. 2, pp. 351–376.

Slemmons, D. B., 1977, “Faults and Earthquake Magnitude,” Miscellaneous Paper 5-73-1, *State-of-the-Art for Assessing Earthquake Hazards in the United States*, Report 6, U.S. Army Corps of Engineers, Washington, D.C.

Slosson, J. E., 1984, “Genesis and Evolution of Guidelines for Geologic Reports,” *Bulletin of the Association of Engineering Geologists*, Vol. 21, No. 3, pp. 295–316.

Soydemir, C., 1991, “Seismic Design of Rigid Underground Walls in New England,” *Proceedings: Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics*, Paper No. 4.6, St. Louis, Missouri.

Stark, T. D., and Mesri, G., 1992, “Undrained Shear Strength of Liquefied Sands for Stability Analysis,” *Journal of the Geotechnical Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 118, No. 11, pp. 1727–1747.

Taner, J. E., 1994, *Structural Calculations—Building 01-080*, Stanford University (unpublished), Stanford, California.

Tokimatsu, A. M., and Seed, H. B., 1987, “Evaluation of Settlements in Sands Due to Earthquake Shaking,” *Journal of the Geotechnical Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 113, No. 8, pp. 681–878.

Tokimatsu, K., and Yoshimi, Y., 1983, "Empirical Correlation of Soil Liquefaction Based on SPT N-Value and Fines Content, Soils and Foundations," *Japanese Society of Soil Mechanics and Foundation Engineering*, Tokyo, Japan, Vol. 23, pp. 56–74.

Utah Section of the Association of Engineering Geologists, 1987, *Guidelines for Evaluating Surface Fault Rupture Hazards in Utah*, Utah Geological Survey Miscellaneous Publication N.

Veletsos, A. S., 1990, "Dynamics of Soil-Structure Interaction Revisited," *Proceedings of Fourth U.S. National Conference on Earthquake Engineering*, Palm Springs, California.

Veletsos, A. S., and Wei, Y. T., 1971, "Lateral and Rocking Vibration of Footings," *Journal of the Soil Mechanics and Foundations Division*, American Society of Civil Engineers, New York, New York, Vol. 97, No. SM9, pp. 1227–1248.

Vesic, A. S., 1975, "Bearing Capacity of Shallow Foundations," *Foundation Engineering Handbook*, edited by Winterkorn, H. F., and Fang, H. Y., Van Nostrand Reinhold, New York, New York, pp. 121–147.

Wallace, J. W., Moehle, J. P., and Martinez-Cruzado, J., 1990, "Implications for the Design of Shear Wall Buildings Using Data from Recent Earthquakes," *Proceedings of Fourth U.S. National Conference on Earthquake Engineering*, Palm Springs, California, pp. 359–368.

Whitman, R. V., 1991, "Seismic Design of Earth Retaining Structures," *Proceedings: Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics*, BiTech Publishers, Vancouver, British Columbia, No. S0A6, pp. 1767.

Wilson, J. C., 1988, "Stiffness of Non-Skewed Monolithic Bridge Abutments for Seismic Analysis," *Earthquake Engineering and Structural Dynamics*, Vol. 16, pp. 867–883.

Winterkorn, H. F., and Fang, H. Y., 1975, *Foundation Engineering Handbook*, Van Nostrand Reinhold, New York, New York.

Wolf, J. P., 1992, "Simple Physical Models for Foundation Vibration - Towards a Strength of Materials Approach," invited paper, Tenth World Conference on Earthquake Engineering, Madrid, Spain.

Wood, J. H., 1973, *Earthquake-Induced Soil Pressures on Structures*, Report No. EERL 73-05, California Institute of Technology, Pasadena, California.

Yegian, M. K., Marciano, E. A., and Gharaman, V. G., 1991, "Earthquake-Induced Permanent Deformations: Probabilistic Approach," *Journal of the Geotechnical Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 117, No. 1, pp. 35–50.

Yim, S. C.-S., and Chopra, A. K., 1985, "Simplified Earthquake Analysis of Multistory Structures with Foundation Uplift," *Journal of the Structural Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 111, No. 12, pp. 2708–2731.

Youd, T. L., 1984, "Recurrence of Liquefaction at the Same Site," *Proceedings, Eighth World Conference on Earthquake Engineering*, Prentice-Hall, Englewood Cliffs, New Jersey, Vol. 3, pp. 231–238.

Youd, T. L., and Perkins, D. M., 1978, "Mapping of Liquefaction Induced Ground Failure Potential," *Journal of the Geotechnical Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 104, No. 4, pp. 433–446.

C5. Steel and Cast Iron (Systematic Rehabilitation)

C5.1 Scope

No commentary is provided for this section.

C5.2 Historical Perspective

This section provides a brief review of the history of cast iron and steel components of building structures. The information was provided through discussions with some structural engineers with decades of experience, examination of plans of older buildings constructed in the early part of the 20th century, review of older steel design textbooks, and review of the *Engineering News Record* and ASCE *Transactions* for the period from approximately 1880 through 1930.

History of Steel Materials and Processes. Iron and steel have been used in the construction of buildings for centuries. Cast iron was first developed as early as 200 BC, and it was produced in significant quantities in the United States during the late 18th century and throughout the 19th century. Cast iron has a relatively high carbon content (more than 1.5%) along with silicon and sulphur. As a result, cast iron is hard and brittle, with limited tensile strength. It is difficult to work, so it must normally be used in cast assemblies. Because of its availability and fairly good compressive strength, it was used quite extensively for columns in buildings built in the early to middle 19th century. Engineers preferred not to use cast iron in components that were either part of a lateral load system or developed significant bending or tension, because of brittle and dramatic failures of cast iron components in bridges and other structures. Cast iron continued to be used into the early part of the 20th century, but wrought iron became the more dominant material in the late 19th century, and steel overtook both in the early 1900s.

Wrought iron was first developed through the hand puddled process in 1613. The metal produced by this process was somewhat variable, depending upon the skill of the producer, and only relatively small quantities of metal could be produced. As a result, this early wrought iron could appear in buildings built before approximately 1850, but it is not likely to be a major structural element because of the small volume that could be produced. Mechanical methods for producing larger quantities of wrought iron were developed in the mid-1800s, and wrought iron was used in the structural systems of a substantial number of buildings in the late 1800s and early 1900s. Wrought iron is much more workable than cast iron; it is more ductile and has better tensile capacity. As a result, it was a more versatile construction material than the cast iron that preceded it. However, for columns, cast iron was still viewed as the most economical material until very late in the 1800s.

Steel was largely made possible by the development of the Bessemer process combined with the open hearth furnace. The Bessemer process was patented in 1856, but steel does not appear to have become commonly available until about 1880. This delay was partly due to some legal disputes, as well as fundamental concerns about the properties and quality of the material. In 1880, wrought iron still dominated the structural market, and buildings built in the mid-1890s were still most likely to be built of wrought iron (possibly with cast iron columns) rather than steel, but most engineers of that period believed that low carbon structural steel was the superior material and would dominate future building construction.

In 1894–95, the first specification for structural steel was published (Campbell, 1895). This document did not address building design, but established quality control and standardization requirements for the material. In 1896, the steel manufacturers agreed to establish some standardization in the shapes that they produced, and steel proceeded to totally dominate the structural market during the next 10 years.

A number of tests for steel and structural steel components are reported during the 1890s. Examination of the reported test results suggests that the properties of this early steel were not very different from the A36 steel used in the 1950s and 1960s. The yield stress may have been somewhat lower, and the early standard designation for this mild steel was A9 with a nominal yield stress of 30 ksi. In the late 1890s fire tests were performed on steel members, and engineers became concerned about fire protection. Masonry was used to enclose the steel and provide fire protection in some early buildings, but it appears that concrete encasement became the predominant form of fire protection at about the start of the 20th century. Riveted connections were the primary method for connecting both wrought iron and steel members during this period.

Steel construction proceeded in a fairly continuous manner in the following years, although there was quite a wide variation in the structures and the materials used in the structures because of particular requirements of the designer. Welding techniques were first developed around 1915 and used in a few structures in the 1920s and 1930s, but usage was limited due to poor quality. Mild steel bolts also had limited usage during this period, and A7 steel with a nominal yield stress of 33 ksi arrived on the scene, essentially replacing A9 by 1940. Further standards for steel and steel products were developed, largely due to the efforts of the American Institute of Steel Construction (AISC), established in the 1920s. This second wave of standardization, with the structural designer involved in the process, resulted in greater uniformity in both the steel and structural steel shapes as well as the structural designs themselves.

Some of the early welding techniques employed gas welding, but electric arc welding was also developed in the very early 1900s. During the 1930s the use of flux and shielding of the arc began. Some structural tests on welded components were performed starting in the 1930s, and electric arc welding became common in the 1940s and 1950s. By the mid-1960s, the use of riveted connections was abandoned as high-strength bolts and electric arc welding became the standard connection technique.

Around this time, concrete encasement for fire protection was also disappearing in favor of lighter insulation methods, and A36 steel with a yield stress of 36 ksi became the standard steel. Higher-strength steels were also introduced during this period.

C5.2.1 Chronology of Steel Buildings

C5.2.1.1 Introduction

Due to the brittle nature of iron, it was not possible to produce shapes by hot or cold working. As a result, iron shapes for columns were cast and often patented.

Some typical shapes are shown in Figure C5-1 (Freitag, 1906). Due to lack of good quality control, cast pieces often had inclusions; this greatly reduced the allowable stress for cast iron columns. A good summary of the use of cast iron in the United States was recently published (Paulson, Tide, and Meinheit, 1994).

As noted in the earlier discussion, cast iron was used extensively throughout the 19th century, but its use was

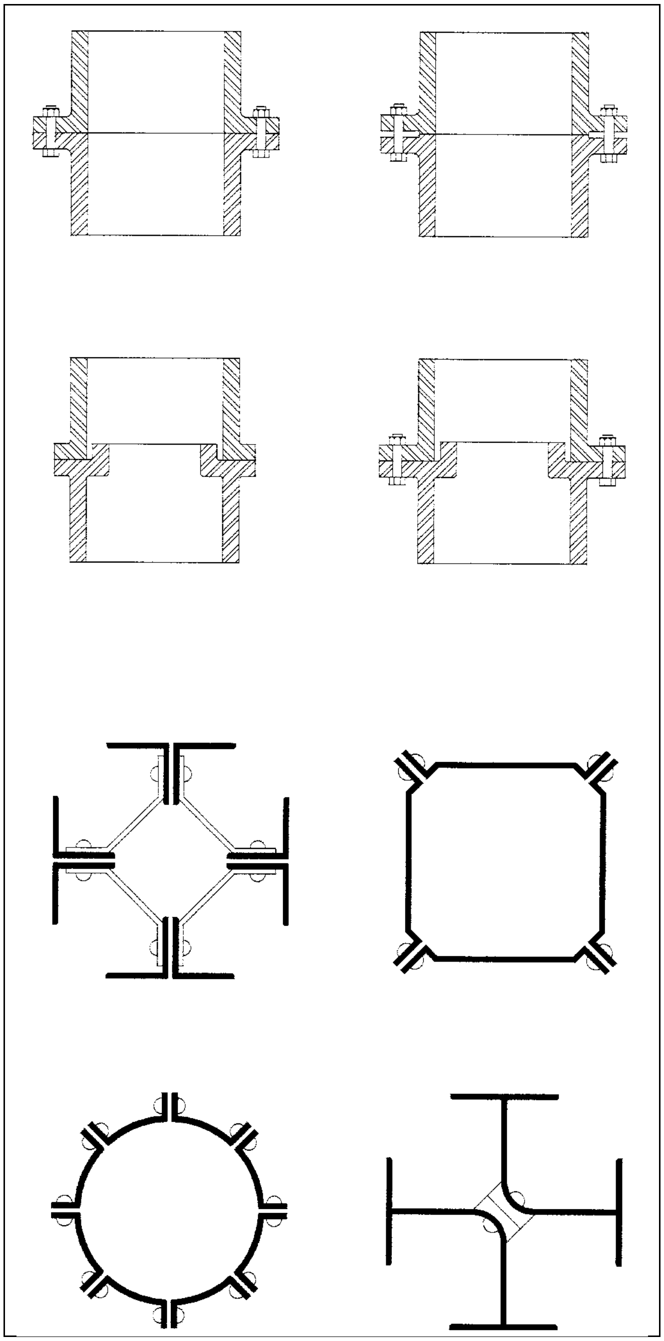


Figure C5-1 Cast and Wrought Iron Column Sections

primarily for columns, which carried compression with no significant tension or bending. Cast iron performed poorly when it was subjected to these alternate stress states, and wrought iron had filled in as an alternate construction material for these other applications in the second half of the 1800s. Wrought iron and cast iron were largely replaced by steel at the turn of the century.

Wrought iron and steel were more ductile than cast iron and more easily worked, and a wide range of field and shop modifications was possible.

These wrought iron and steel buildings had some common attributes, but in general, the members and connections were unique. Engineers made extensive use of riveted built-up steel and wrought iron members with riveted connections. The members were commonly built up from plates, angles, and channels. These built-up members used tie plates and lacing, and the large number of rivets made them labor-intensive. Connections were formed with haunches, knee braces, and large gusset plates. The first effort to standardize the steel materials and shapes was made in about 1895, but there was relatively little standardization in design. Each engineer would use his own unique member and connection configurations. Further, the design was controlled by local practice and city building codes. As a result, the predicted strength of the member varied widely. An article published in the mid-1890s illustrates this, noting that one column of a given material and geometry could support 100 tons in New York City, 89 tons in Chicago, and only 79 tons in Boston. These local building codes played a role in restricting the use of wrought iron over steel in many cities, and this contributed to the fuzzy transition between the two materials.

The first proposed structural design specification for steel buildings was published by ASCE (Schneider, 1905). This article examined the wide variation in design loads and stress limits, and proposed a standard design procedure, which began to become a reality with the development of the AISC specification and design manual in the 1920s.

While the members and connections were quite variable, there was a lot of similarity in the general structural aspects of these older buildings. First, they usually had massive fire protection. Massive—but lightly reinforced—concrete was used in most buildings constructed after 1900. The concrete was relatively low-strength and often of questionable quality. In addition, these buildings usually had unreinforced masonry for outside walls, and unreinforced clay tile or masonry partitions throughout the interior. These walls and partitions provide the bulk of the strength and stiffness of these older buildings for resisting lateral loads. These buildings were normally designed for wind load but not seismic loading. They were designed as moment frames, with the tacit understanding that infilled walls help to resist lateral loads but do so without any design calculations.

To illustrate further the variability of construction in this era, it should be noted that engineers readily and quickly shifted from one material to another. Concrete encasement was not considered in the evaluation of the strength of steel structures, but it was readily used as a transition between steel and concrete construction. Some engineers shifted from steel to concrete columns, or they connected a reinforced concrete beam to a steel column or beam, and used the encasement for the development of the two different members.

C5.2.1.2 1920 through 1950

In the 1920s, use of the unique, complex built-up members began to be phased out, and standard I and H shapes replaced them as the standard for member design. Partially restrained (PR) connections, such as the riveted T-stub and clip angle connections discussed in Section C5.4.3.3, became the normal connection. Because the clip angle connections were weaker and more flexible, they were used as the beam column connections in shorter buildings or in the top stories of taller buildings. The T-stub connection was stiffer and stronger, and it was used in the lower floors of taller buildings where the connection moments were larger. Stiffened angle or T-stub connections were often used to provide a beam connection to the weak axis of the column.

Lightly reinforced concrete was still used for fire protection. The concrete was sometimes of higher strength, but still often of questionable quality. Unreinforced masonry was still used for outside walls and unreinforced clay tile for masonry partitions throughout the building. Buildings constructed in regions regarded as seismically active were designed for seismic forces, but the design forces were invariably lower than those required today. However, the walls and partitions were not included in the design calculations, and they still provided the bulk of the strength and stiffness of these buildings. Buildings outside of regions of known seismic activity were designed for wind load only.

It should be noted that all buildings constructed during this era used relatively simple design calculations compared to modern buildings. Engineers frequently resorted to observations from past building performance and standard practice; the sophisticated computer calculations used in modern structures were unknown.

Bolts and welding were sometimes used, but rivets were clearly the dominant connection. They were designed as moment frames, but actual structural behavior was strongly influenced by stiff, strong masonry infills and partitions.

C5.2.1.3 1950 through 1970

Significant changes began to appear during this period. The use of rivets was discontinued in favor of high-strength bolts and welding. In the very first structures, bolts were merely used to replace the rivets in connections such as the clip angle and T-stub connection illustrated in Figure C5-2. However, flange plate and end plate connections, such as those discussed in Section C5.4.3.3, were used more frequently. Increased use of and confidence in welding made these connections possible. By using these connections, engineers were often able to develop greater connection strength and stiffness with less labor. Another important change was the replacement of standard concrete fire protection by more modern lightweight materials.

Two more changes are notable. For one, masonry and clay tile walls were less frequently used for cladding and partitions, reducing building weight, although the architectural elements were still significantly heavier and stiffer than those used in steel frames today. However, these panels and finishes were more likely to be attached to the structure rather than being used as an infill to the frame. As a result, buildings built during this era are sometimes less able to utilize this added strength and stiffness than are the older structures. Finally, significant differences began to evolve in the way buildings were designed for regions of high seismic activity, and for other regions. These regional differences were developed because regions with significant seismic design requirements had to deal with larger lateral forces, but also because of the increased emphasis on ductility in seismic design procedures. In less seismically active zones, the weaker, more flexible connections were retained for a longer period of time, while in the seismically active zones the fully restrained FR connection discussed in Section C5.4.2 began to evolve. Also, braced frames and alternate structural systems were used because they could often achieve much greater strength and ductility with less steel and more economical connections.

C5.2.1.4 1970 to the Present

The trends established in the 1960s continued into the following period. First, there was increased emphasis

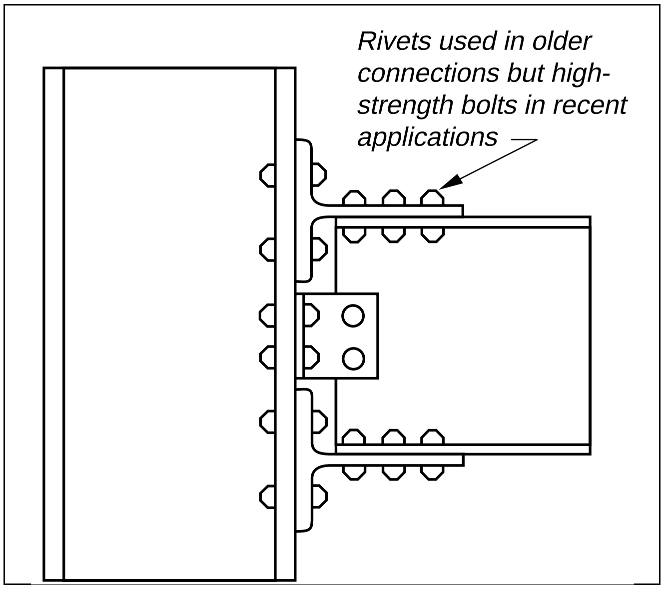


Figure C5-2 Riveted T-Stub Connection

on lightweight fire protection and architectural elements. As a result, the reserve strength and stiffness provided by these elements was reduced.

Second, there was increased emphasis on ductility in seismic design, and extensive rules—intended to assure ductility for moment frames, braced frames, and other structural systems—were established. These rules undoubtedly had some substantial benefit, but compliance was often expensive, and there was a distinct tendency toward using structures with less redundancy, since these less-redundant structures required satisfaction of the ductility criteria at fewer locations. This reduced redundancy also resulted in larger member and connection sizes. This separation of the practice between regions with significant seismic design requirements, and those with little or no seismic design requirements, continued to widen. The less seismically active regions sometimes retained more flexible connections with greater redundancy in the overall structure.

Third, seismic design forces were appearing for the first time in many parts of the United States, and they increased significantly for all parts of the country for some structural systems. Finally, the steel and construction processes themselves were also changing. There was a significant increase in steel produced by reprocessing scrap metal in an electric furnace. As a result, the yield stress of standard steels increased, while the tensile stress remained relatively stable.

Welding evolved from the relatively expensive stick welding shielded arc process to the quicker and more economical flux core, gas shield, and dual shield processes. High-strength bolts were increasingly used as slip-critical friction bolts; however, quality control variations caused by tightening and installation became a major concern. These changes in turn produced changes in the ductility and behavior of many steel structures.

C5.2.2 Causes of Failures in Steel Buildings

Until quite recently, major failures in steel components and buildings were rare. Five steel buildings collapsed or were fatally damaged in Mexico City during the 1985 Michoacan earthquake. This damage was the result of a large torsion irregularity, a resonance condition between the soft soil and the building, and, perhaps, poor fabrication of the built-up square columns. Other typical damage include buckled braces, failure of a few connections, and damage to infills and attached cladding. Loss of entire masonry cladding from entire sides of a building was observed.

Prior to the 1994 Northridge, California earthquake, the steel moment frame was considered to be the ideal structural element to resist earthquakes because of its excellent ductility. However, during this earthquake over two hundred buildings experienced fractured beam-column or column-baseplate connections. The reasons for this poor performance are complex, and still under investigation. One significant factor was lack of quality control of the entire welding process, in combination with the use of weld filler that has almost no notch toughness. Other factors contributed to this poor behavior, such as the thickness of the column and beam flanges, the stiffness and strength of the panel zones, triaxial stress effects, high confinement of the joints, and poor welding procedures, for example, high heat input, rapid cooldown, and conditions allowing hydrogen embrittlement. A discussion of the different types of fractures and ways of preventing or repairing them is given in FEMA 267 (SAC, 1995). The increased beam depths used in current designs also played an important role (Roeder and Foutch, 1996), along with poor quality in construction.

C5.3 Material Properties and Condition Assessment

C5.3.1 General

No commentary is provided for this section.

C5.3.2 Properties of In-Place Materials and Components

C5.3.2.1 Material Properties

No commentary is provided for this section.

C5.3.2.2 Component Properties

Identification of critical load-bearing members, transfer mechanisms, and connections must be established on the basis of a review of available data. It is often possible to classify structural member types—whether rolled or built-up—and material grade and general properties, by examining the original building drawings and construction documents. Local verification of matching members and materials to the construction documents is necessary in order to examine any gross changes that may have occurred since construction began. If these drawings and documents are not available, the subject building’s components must be determined (e.g., size, condition), and the material type(s) identified.

C5.3.2.3 Test Methods to Quantify Properties

A variety of building material data is needed for conducting a thorough seismic analysis and rehabilitation design. For metallic structures, which are often enclosed or encased in the architectural fabric, these needs range from verification of physical presence to specific knowledge of material properties, member behavior, connection details and type, and condition. Many buildings have been structurally altered during their service life existence, without corresponding drawing updates or other notification. Verification of gravity and lateral-load-resisting members and their connection configuration is essential.

After member and connection presence and types are confirmed, mechanical properties must be quantified. The amount of effort needed to establish properties varies considerably, depending on the availability of building drawings and data. Several common steps may be taken to gain confidence regarding the materials used and their properties. These steps, in preferred order, include:

- Retrieval of building drawings, specifications, improvement records, and similar information
- Definition of the age of the building (e.g., when the building materials were procured and erected)
- Comparison of age and drawing information to reference standards
- Field material identification with in-place nondestructive testing
- Acquisition of representative material samples from existing members and performance of laboratory mechanical tests (e.g., tensile, offset yield, impact, chemical)
- Performance of in-place metallurgical tests to determine the relative state of the crystalline structure and presence of structural damage

Finally, the physical condition of the structural system must be examined to determine whether defects are present that would prevent any member from performing its function. For accessible members and connections, visual inspection should be performed for condition assessment. Other methods for quantifying the physical condition of a structure are specified in the *Guidelines*, Section 5.3.2.

A wide range of evaluation methods and tools exists for verifying the existence, and determining the mechanical properties and physical condition, of a metallic building element. Also, many reference standards for material behavior are given in the following reference standards for metallic structures:

1. American Institute of Bolt, Nut and Rivet Manufacturers (defunct)

   Tentative Specifications for Cold Riveted Construction

2. American Institute for Hollow Structural Sections (formerly Welded Steel Tube Institute)

   Structural Steel Tubing

3. American Institute for Steel Construction (AISC)

   Manual of Steel Construction

   Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings

   AISC Iron and Steel Beams, 1873 to 1952

4. American Iron and Steel Institute (AISI)

   Specification for the Design of Cold-Formed Steel Structural Members

   Specification for the Design of Cold-Formed Stainless Steel Structural Members

   Sectional Properties of Corrugated Steel Sheets

   AISI Standard Steels

   Fastening of Lightweight Steel Framing

   Load and Resistance Factor Design Specification for Cold-Formed Steel Structural Members

5. American Society for Metals (ASM)

   "Properties and Selection: Irons, Steels and High-Performance Alloys," *ASM Handbook, Volume 1*

   "Nondestructive Testing and Quality Control," *Metals Handbook, 9th Edition, Volume 8*, 1992

   "Failure Analysis and Prevention," *Metals Handbook, 10th Edition, Volume 10*, 1989

   "Corrosion," *Metals Handbook, Ninth Edition, Volume 13*, 1987

   "Nondestructive Testing and Quality Control," *Metals Handbook, Volume 17*, 1989

   "Metallography and Microstructures," *ASM Metals Handbook, Volume 9*, 1985

6. American Society of Mechanical Engineers (ASME)

   Bibliography on Riveted Joints

7. American Society of Civil Engineers (ASCE)

   "Specification for the Design of Cold-Formed Stainless Steel Structural Members," *ANSI/ASCE 8-90*

   Bibliography on Bolted and Riveted Joints (Manual 48)

   "Guideline for Structural Condition Assessment of Existing Buildings," *ASCE Standard 11-90*, 1991

8. American Society for Testing and Materials (ASTM)

   Annual Book of Standards (material specifications for base metals and all forms of connector material)

   "Standard Practice for Measuring Thickness by Manual Ultrasonic Pulse-Echo Contact Method," *ASTM E797-87*, 1987

   "Metals—Mechanical Testing; Elevated and Low-Temperature Tests; Metallography," *Annual Book of Standards, Volume 03.01*, 1993

   (Particular emphasis on Designations A370, E8 [tensile], E9 [compression], E10/18 [hardness], E110 [portable hardness], E290 [ductility], and E399 [fracture toughness])

9. American Welding Society

   Structural Welding Code—Steel, AWS D1.1

   Code for Arc and Gas Welding in Building Construction

   Filler Metal Specifications

10. Industrial Fasteners Institute (IFI)

    Fastener Standards

11. International Standards Organization

    Steel Construction—Materials and Design

12. Research Council on Riveted and Bolted Structural Joints of the Engineering Foundation

    Specifications for Assembly of Structural Joints Using High-Strength Bolts

    Specification for Structural Joints Using ASTM A325 or A490 Bolts (Allowable Stress Design and Load and Resistance Factor Design)

13. Steel Deck Institute (SDI)

    SDI Design Manual for Composite Decks, Form Decks and Roof Decks

14. Steel Joist Institute (SJI)

    Standard Specifications, Load Tables and Weight Tables for Steel Joists and Joist Girders

    50 Year Steel Joist Digest

15. United States Department of Commerce, National Institute of Science and Technology (formerly National Bureau of Standards)

    Simplified Practice Recommendation R-216-46 (discontinued)

16. Welded Steel Tube Institute (now American Institute for Hollow Structural Sections)

    Welded Carbon Steel Mechanical Tubing

    Dimensions and Properties of Cold Formed Welded Structural Steel Tubing

C5.3.2.4 Minimum Number of Tests

The material testing requirements described in the *Guidelines* should be considered as a minimum. Where construction documents and drawings are not available, the design professional must insist that some inspection and material testing be done if the evaluation and rehabilitation is to proceed. This must be done even if removal and replacement of architectural features results in some inconvenience to the occupants.

ASTM Designation A370 contains standard test methods for determining tensile, bend, impact, and hardness properties of steel and iron elements. Testing of in situ materials may be done on smaller specimens than those described in A370, but the dimensions must be scaled down proportionately. Included in this specification, ASTM Designations E9 and E11 provide procedures for computing compressive strength and Young's, tangent, and chord moduli.

C5.3.2.5 Default Properties

For older buildings where steel components are encased in concrete, or for buildings with great historical importance, it may be prohibitively expensive to do all of the testing required by one of the nonlinear procedures. A lesser amount of testing may be done if it is supplemented with additional analysis. The upper and lower bounds on component force demands must be estimated. The first analysis should be done using the minimum strength values determined through testing, supplemented by default values. A second analysis must be done where lower bound material strengths are used for columns and connections and upper bound material strengths are used for braces and beams. The upper bound strengths should be 30 to 50% greater than the default values given in Tables 5-1 and 5-2.

C5.3.3 Condition Assessment

C5.3.3.1 General

Establishing the physical presence of metallic structural members in a building may be as simple as direct visual inspection and measurement, or as complex as using gamma radiography (through the architectural fabric) or boroscopic review through drilled access holes—methods that may be necessary if access is not permitted. The survey should include both base element and connector materials and details. For elements encased in concrete or fireproofing, this verification may be done by removing such encasements at critical locations.

It is well recognized that metallic components degrade if exposed to an aggressive environment. Corrosion is especially degrading in terms of lost material, reduction of properties, and propensity for creating locally embrittled areas. Assessment of in-place physical condition may be accomplished through visual inspection, nondestructive testing (NDT), and sampling and destructive testing techniques. Quantification of condition may consist of taking ultrasonic material thicknesses for comparison to original/nominal thickness, comparing existing material response to sound and vibration to that of new (calibrated) material, or using recently developed tomographic methods. Depending on the physical conditions of the element/connections, the number of tests necessary to gain confidence will vary substantially. Recommended guidelines for visual condition assessment are contained in ASCE Standard 11-90 for both base metals and connectors. Of particular interest during the survey are any existing conditions not reflected in the design documents (e.g., different end connectors), presence of any degradation, integrity of any surface coatings, and signs of any past movement.

Visual inspection of weldments should be made in accordance with American Welding Society D1.1, "Structural Welding Code—Steel." Structural bolts should be verified to be in proper configuration and tightened as required in AISC's *Steel Construction Manual*. Rivets should also be verified to be in proper configuration and in full contact, with "hammer sounding" conducted on several random rivets to ensure that they are functional.

Other nondestructive testing methods that may be used include liquid penetrant and magnetic particle testing (weld soundness), acoustic emission (system and element behavior), radiography (connector condition), and ultrasonics (numerous uses). Nondestructive testing should be used when visual inspection identifies ongoing degradation, or when a particular element or connection is critical to seismic resistance and requires further verification. Information on these methods and descriptions of their application are contained in a number of references.

It is recommended that all critical building elements be visually inspected, if possible, based on access and available time.

C5.3.4 Knowledge (κ) factor

No commentary is provided for this section.

C5.4 Steel Moment Frames

C5.4.1 General

Steel moment frames are categorized by the connection type. The connections vary widely between modern welded connections with high-strength bolts, and older riveted connections with gusset plates, angles, and T-sections connecting standard rolled shapes and complex built-up members. Modern connections with welded flanges and bolted webs deform and rotate very little, and are regarded as fully restrained (FR) connections.

Partially restrained (PR) connections develop significant rotation and deformation within the connection. Many riveted and bolted connections qualify as PR connections, but the connection strengths and stiffnesses vary widely. Figure C5-3 shows the relative deformability and stiffness of different connections.

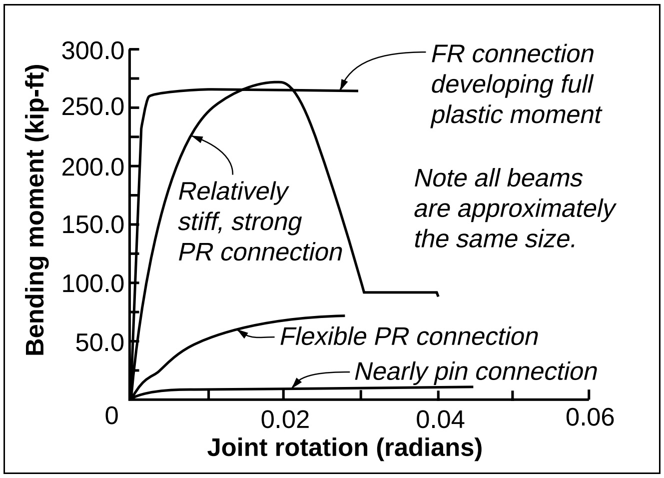


Figure C5-3 M-θ Relationships for FR and PR Connections

C5.4.2 Fully Restrained Moment Frames

C5.4.2.1 General

Fully restrained (FR) moment frames have nearly rigid connections. The connections must be at least as strong as the member, and the deformation of the connections can contribute no more than 5% of the story drift. Special Moment Frames are typically designed for small seismic forces, because they dissipate large quantities of energy through flexural yield of beams and columns or shear yield of the panel zone. As a result, local flange and web buckling and lateral torsional buckling of beams and columns of Special Moment Frames must be controlled in the hinging regions, even for end rotations as large as four to six times the rotation at yield. Ordinary Moment Frames must also meet limited ductility requirements, but the plastic end rotation requirements are smaller, and the slenderness limits for the web, flange, and lateral torsional buckling are less severe. The terms Ordinary and Special Moment Frames are not used in the *Guidelines*, but the limits used in the *Guidelines* are based on limits associated with these two moment frames in other documents, such as AISC (1994a).

FR moment frame members that are encased in concrete for fire protection are unlikely to experience the deformation associated with local buckling that is encountered with bare steel frames. This prevents the deterioration associated with local buckling, and allows the steel to develop its full ductility and yield capacity without the many local stability concerns outlined in AISC (1994a). As a result, these encased frames are assumed to satisfy the requirements of Special Moment Frames.

Special Moment Frames historically had a very good reputation for ductility and seismic performance, but because a significant number of these frames experienced cracking in the 1994 Northridge earthquake, special provisions are included in this document.

C5.4.2.2 Stiffness for Analysis

The stiffness and the resulting deflections and dynamic period of FR moment frames are determined by the usual structural analysis procedures. The contributions of elastic deformation of the connections to frame deflection are not addressed, because these contributed frame deflections are relatively small compared to deflections caused by member deformations. Elastic stiffness is dependent upon the geometric properties of the members; for modern steel frames with lightweight fire protection, these are the properties of the bare steel section. For older steel frames that are encased in concrete for fire protection, composite member properties should be used for elastic analysis if the concrete is in contact with the steel. This increased stiffness may be very significant, and can lead to larger seismic forces.

During inelastic analysis, changes in incremental stiffness occur due to yielding, and the inelastic stiffness is therefore interrelated with the strength. FR moment frames yield in the beams, columns, and panel zones during inelastic deformation. Stiffness must be reduced at these locations when yielding occurs. Computer models such as those developed for PR connections and described in Section C5.4.3.2 are sometimes used to approximate panel zone yield deformation. While the stiffness is reduced for yielded members and panel zones, the elastic stiffness is still used for all other members and connections.

The yield deflections and strength rules included in Section 5.4.2.2 are based on typical plastic design models such as those used in the AISC LRFD

Specification (AISC, 1994b). The yield deflections for beams and columns are based on conservative approximations. The true frame deflection at initiation of significant yielding may be slightly larger than predicted, and as a result, the true ductility demand should be somewhat smaller than predicted by these guidelines. This conservative procedure is based on the assumption of cantilevered members with inflection points at mid-height of the column and mid-span of the beam. The method further assumes that the rotation all occurs in the most flexible element. The members are assumed to remain elastic until the full plastic moment is developed. The plastic moment capacity for members under combined loading is adjusted for the axial load by linear interpolation.

C5.4.2.3 Strength and Deformation Acceptance Criteria

The significant deformation given in Table 5-4 is plastic end rotation. This was chosen to be consistent with the concrete chapter, and because some popular computer programs give plastic end rotation as standard output. The majority of test results give chord rotation, which is depicted in Figure 5-2, as the deformation response. There is little actual difference between the two for large deformations. The chord rotation may be estimated as the plastic end rotation plus the yield rotation.

The strength of individual members and components is defined by plastic analysis techniques, except that linear interpolation is sometimes used for transitions between one established condition and another.

Composite action due to concrete encasement is not considered in the resistance, because the bond stress or shear transfer mechanism is important to member behavior, and the condition of this interface is uncertain in existing structures. Further, the additional strength contributed by composite action of FR moment frames often is relatively small. While the strength provided by encasement is not factored in, the stiffness provided to the steel by the concrete is considered.

A. Linear Static and Dynamic Procedures

There is no strict story drift limit for steel frames. For the Immediate Occupancy Performance Level, a drift level less than 0.01 is desirable. This limit is selected because steel frames normally experience their first significant yielding at an inter-story drift ratio of between 0.005 and 0.010. Steel is a ductile material and no significant damage is expected at the 0.01 drift level. Practical drift limits for Life Safety and Collapse Prevention performance might be 0.02 and 0.04, respectively.

Significant inelastic deformation is permitted in ductile elements for the Life Safety and Collapse Prevention Performance Levels. Collapse Prevention *m* values by definition represent maximum permissible post-yield deformation for components based on the Collapse Prevention limit state. They are to be specified for each type of component, recognizing the types of forces (axial, shear, flexure) and considering the mode of failure. Table 5-3 indicates the components to be covered. When using the linear procedures, *m* factors reduce the seismic design forces because of inelastic behavior and component ductility. Good inelastic performance indicates good energy dissipation and the ability of the component to hold together through significant inelastic deformations. For Life Safety, *m* values are invariably smaller than *m* values for Collapse Prevention because the Life Safety limit state can tolerate less damage to the structure.

Historically, Special Moment Frames have been regarded as very ductile structural systems that can tolerate plastic deformations on the order of four times the yield deformation with little or no deterioration in strength or ductility. Larger inelastic deformations are possible if some deterioration is tolerated. Ordinary Moment Frames are somewhat less ductile. The Collapse Prevention *m* values given for beams and columns in moment frames in Table 5-3 are based upon member behavior. The more restrictive limits on frame properties with larger *m* values are based upon AISC (1994a) limits for Special Moment Frame behavior. The least restrictive limits on frame properties with smaller *m* values are based upon Ordinary Moment Frame behavior. Interpolation is allowed between these extreme limits; however, it must be emphasized that these are member ductility limits, and separate limits are applied to the connections of FR steel moment frames.

A number of FR steel moment frames experienced cracking in the joints and connections during the Northridge earthquake. As a result, the *m* values for FR moment frame connections are evaluated separately in Table 5-3. This evaluation was achieved by examining the results of more than 120 experiments on FR moment connections under inelastic cyclic loading, all performed in the United States in the past 30 years (Roeder and Foutch, 1996). This evaluation clearly showed that the flexural ductility achieved with FR moment frame connections is dramatically reduced with deeper beams. The empirically determined equation,

$$m = 7.5 - 0.125\ d_b \qquad \text{(C5-1)}$$

is based on a least squares fit to experimental results. This equation has been slightly reduced for safety for use with the *Guidelines*. The term, d_b, is the beam depth. This same experimental data showed that flexural ductility is significantly reduced in beams with panel zone yielding. This occurs because of the severe local deformation occurring near the welded connection with panel zone yield deformation. The ductility achieved with the panel zone itself may be very large, but there is significantly larger strain hardening with shear yield of the panel zone than with flexural yielding. As a result, the bending moments in the welded connection grow significantly larger during panel zone yielding, and the second set of connection limits is provided.

B. Nonlinear Static Procedure

The NSP uses a nonlinear pushover analysis to evaluate inelastic behavior. The deformations permitted in each element utilize a logic that is very close to that employed in the evaluation of *m* values. Table 5-4 defines the deformation limits for FR moment frames.

C. Nonlinear Dynamic Procedure

The deformation limits provided in Table 5-4 also apply to the deformations achieved in the NDP.

C5.4.2.4 Rehabilitation Measures for FR Moment Frames

A. Component Strength Enhancement Techniques

- **Columns**
  - Shear capacity—Add steel plates parallel to web (doubler or at flanges) or encase in concrete.
  - Moment capacity—Add steel plates to flanges or parallel to web, or encase in concrete.
  - Axial—Add steel plates or encase in concrete.
  - Combined—See above.
  - Stability—Provide steel plates, stiffeners, bracing members, or concrete encasement.
  - Strong column-weak beam—Strengthen column using techniques noted above.
  - Concrete encasement—Remove or modify in cases where concrete causes potential undesirable failure mode.
- **Beams**
  - Shear—Add steel plates parallel to web (doubler or at flanges) or encase in concrete. These are probably only needed over a certain length adjacent to connections.
  - Moment—Add steel plates to both flanges, bottom flange only (if composite action is reliable), or beam encasement, or augment composite slab participation. Effects on strong column-weak beam conditions should be considered. Again, these are probably only needed over a certain length adjacent to connections.
  - Stability—Provide lateral bracing for unsupported flange(s) (usually only the bottom flange, since the top flange is braced by the concrete diaphragm) with perpendicular elements or stiffeners. Both strength and stiffness need to be considered.
  - Concrete encasement—Remove or modify encasement or composite action where they create potential undesirable failure modes.
- **Connections**
  - Beam flange to column—The choice depends on the type of connection. For fully welded connections, modify in accordance with FEMA 267 (SAC, 1995). For flange plates, add plates, and/or welding.
  - Beam to column web—Add welding; replace rivets with high-strength bolts.
  - Concrete encasement—Remove or modify encasement or composite action where it creates potential undesirable failure modes.
  - Column base fixity—Add anchor bolts; add welding; add stiffening plates to column and base plate.

- **Joints**
  - Panel zone shear strength—Add doubler plates with various details.
  - Column flange stiffness—Add continuity plates or stiffen flanges with additional plates.
  - Column web crippling—Add continuity plates and/or doubler plates, or concrete encasement.
  - Column web tearing—Add continuity plates and/or doubler plates, or concrete encasement.

B. Rehabilitation Measures for Deformation Deficiencies

Almost all member-strengthening techniques will also enhance member stiffness. The amount of stiffening can vary substantially depending on the technique. Only minor stiffening will result from additional welding, replacement of rivets, or addition of continuity plates; moderate stiffening from addition of steel plates, or augmentation of composite action; and the most substantial stiffening from concrete encasement. Effects on frame strength and failure modes must be considered.

C. Connection Between New and Existing Components—Compatibility Requirements

- **Within Component**

  When choosing rehabilitation measures, the following compatibility requirements apply to connections between new and existing components.

  - Built-up steel sections—Consider the load transfer mechanism between pieces of built-up section (stitch or lacing plates) by welding, bolting, or riveting as it affects strength and stiffness, both elastic and cyclic.
  - Composite beam elements—Consider the interaction of steel beam and concrete slab, the load transfer mechanism (shear connectors or puddle welds), and the effects of both on element strength and stiffness, both elastic and cyclic.
  - Concrete encasement—Consider the interaction of concrete and steel, the load transfer mechanism (friction or shear connectors), and the effects of both on element strength and stiffness, both elastic and cyclic.

- **Within Frame**
  - Connection stiffness and strength—Connection size (especially older systems) may alter frame response, increasing stiffness by reducing clear member lengths. Weak connections limit the load to frame elements.
  - Joint stiffness and strength—Weak joints limit the load to frame elements, but may cause local stress concentrations (column flange kinking).

- **Between Frame and Other Vertical Lateral-Force-Resisting Elements**
  - Stiffness compatibility—Consider the frame/wall effect in tall structures (reverse shears in walls or braced frames at upper stories).
  - Collector/drag elements—The method of distribution of loads to elements should be considered.

- **Interaction with Diaphragm Stiffness**
  - Load distribution—Consider whether rigid versus flexible diaphragms.
  - Load transfer mechanism—Consider mechanisms such as collectors/drags, shear connectors, puddle welds, friction, and bearing, and their effects on strength and stiffness.
  - Diaphragm yielding mechanism—Consider limit load to frames, and the effect on local drifts.

D. Connections in FR Frames

Connections in FR frames must be at least as strong as the weaker member being connected. Rigid connections are commonly used in modern seismic design, and the procedures for dealing with them are documented in other references. Full-pen beam-to-column connections performed poorly during the 1994 Northridge earthquake. Enhancement techniques are given in FEMA 267 (SAC, 1995).

C5.4.3 Partially Restrained Moment Frames

C5.4.3.1 General

Partially restrained (PR) moment frames are those steel moment frames in which the strength and stiffness of the frame is dominated or strongly influenced by the strength and stiffness of the connection. Because of this, the connection strength, M_{CE}, and the rotational spring stiffness, K_θ, are important considerations. In FR moment frames, the analysis of the frame is performed with the assumption that the originally undeformed angle between connected members is retained during seismic deformation. This assumption is not valid with PR connections. Typical moment-rotation relationships for FR and PR connections are depicted in Figure C5-3. Finite element analyses that include the rotational springs as well as the stiffness of the beams and columns must be performed as depicted in Figure C5-4, where K_s is the spring stiffness.

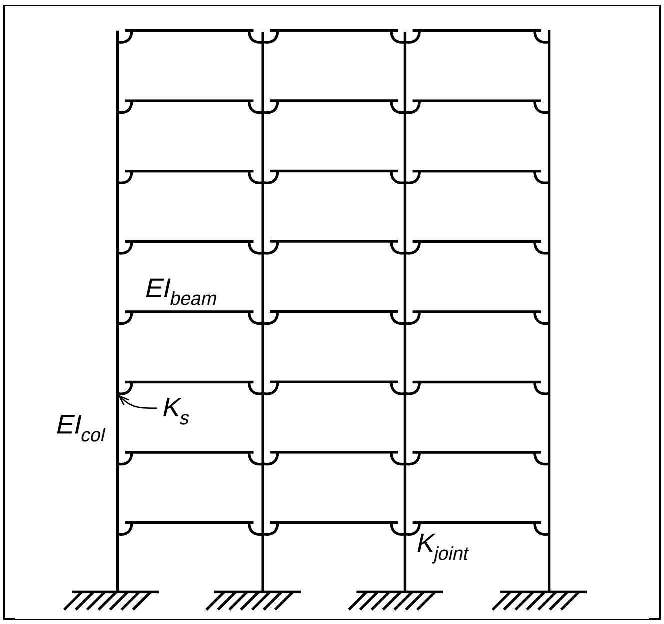


Figure C5-4 Model of PR Frame

While the strength and stiffness of PR connections are limited, many PR connections can sustain very large deformations without failure of the connection or structure. Experimental research has shown that the joint rotation of the connection is an important limiting factor for Life Safety and Collapse Prevention. Therefore, the joint rotation, θ, of each joint due to the application of the unreduced seismic loading must be determined as part of the nonlinear structural analysis. This maximum rotation is then compared to the rotation limits in Table 5-6 of the *Guidelines*. Typical hysteresis behavior of PR connections is shown in Figure C5-5.

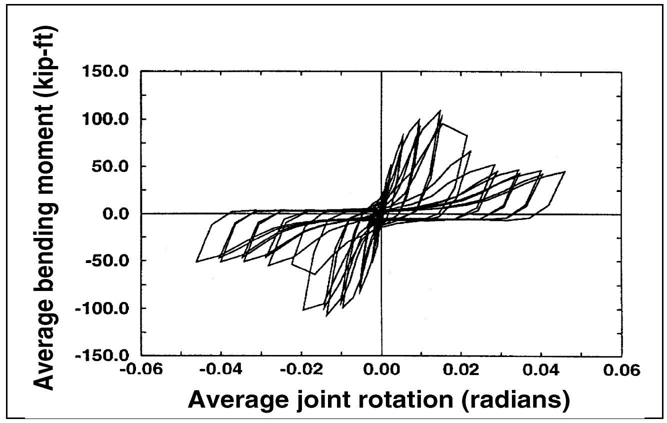


Figure C5-5 Hysteresis of PR Connection

C5.4.3.2 Stiffness for Analysis

The rotational spring stiffness, K_θ, is an important part of the structural analysis of frames with PR connections. However, experimental research has shown that the connection stiffness varies widely based on parameters such as connector size and type, thickness of steel elements, and depth of beam. Composite action due to concrete encasement also significantly increases the stiffness of some connections. The tangent modulus stiffness and the secant modulus stiffness also decrease with increasing joint rotation. Empirical models have been developed for a range of connection types, but these models are inexact and do not cover the full range of connections provided. The simplified models used in this document are based on the experimental observations that connections that are stronger are usually also stiffer. All PR connections experience significant yield and reduction of stiffness at joint rotations on the order of 0.005 radians. A realistic estimate of connection strength is essential to the seismic evaluation and rehabilitation of these structures, so the approximate connection stiffness in Equation C5-2 is employed. That is,

$$K_\theta = \frac{M_{CE}}{0.005} \qquad \text{(C5-2)}$$

Section 5.4.3 provides guidance in evaluating the connection strength, M_{CE}, used to approximate the stiffness. The rotational spring stiffness provided by Equation C5-2 is invariably an intermediate stiffness. It is smaller than the maximum stiffness at zero load, and much larger than the tangent stiffness at failure. This stiffness is needed to establish the initial dynamic period and the seismic forces of the structure.

Composite action due to encasement for fire protection dramatically increases both the strength and stiffness of some PR connections. The engineer has the option of including this additional resistance in the calculation of M_{CE}, but this calculation is more difficult and requires additional effort. In the absence of this added effort, the simplified resistance calculations provided in this document are believed to be conservative. Therefore, the engineer has the conservative option of neglecting this extra resistance in making the design calculations. It is essential, however, that the engineer not neglect the added stiffness, since this would result in a potentially nonconservative underestimate of the seismic forces. Therefore,

$$K_\theta = \frac{M_{CE}}{0.003} \tag{C5-3}$$

is proposed for the special case where the connection is encased and develops composite action. The composite action is neglected in the connection strength calculation.

The rotational spring stiffness is important, but relative frame stiffness determines whether the frame has PR or FR connections. It is preferred that a computer model with frame elements and rotational spring elements, as illustrated in Figure C5-4, be used in determining the frame stiffness. However, many engineers and structural analysis computer programs are not able to easily accommodate the rotational spring. Therefore, a simplified analysis method is proposed as an alternative to a full PR frame analysis. This alternative method allows an analysis with rigid connections, but the beam stiffness, EI_b, is reduced to $EI_b\,adj$—adjusted to account for the rotational spring stiffness of the joint. This adjusted stiffness may be substituted in an ordinary rigid-connection frame analysis.

The fundamental assumptions of the adjusted model are based on the simple single-story moment frame subassemblage illustrated in Figure C5-6. This frame has rigid connections with a bending stiffness EI for the beams and columns; an average beam span length, l_b; and an average story height, h. The centerline member lengths are used, and panel zone rigidity is neglected.

The elastic story drift-deflection, u, can be estimated by the equation

$$u = \frac{Ph^3}{12EI_c} + \frac{Phl_b^2}{12EI_b} \tag{C5-4}$$

where

h = Story height, in.

l_b = Beam length, in.

I_b = Moment of inertia of beam, in.^{4}

I_c = Moment of inertia of column, in.^{4}

It can be seen that the deflection is made up of two parts: bending of columns and bending of beams. If the loads and beam and column stiffness are unchanged, the moment and beam curvature are unchanged, and the story drift deflection for a frame with flexible connections becomes

$$u = \frac{Ph^3}{12EI_c} + \frac{Phl_b^2}{12EI_b} + \frac{Ph^2}{2K_S} \tag{C5-5}$$

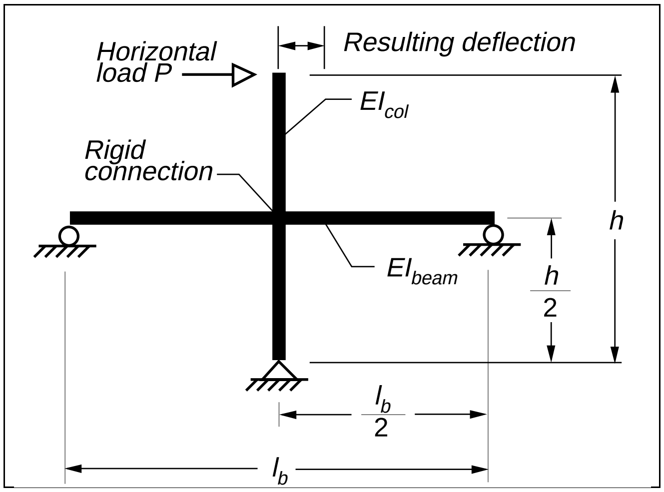


Figure C5-6 Frame Subassemblage

As indicated, a third term is added to this frame deflection based on the rotational spring stiffness of the connection. The simplified model allows the engineer to use Equation C5-4 to achieve the same deflection as achieved with Equation C5-5, that is,

$$u = \frac{Ph^3}{12EI_c} + \frac{Phl_b^2}{12EI_b adj} \tag{C5-6}$$

where

$$EI_b adj = \frac{1}{\dfrac{6h}{l_b^2 K_\theta} + \dfrac{1}{EI_b}} \tag{C5-7}$$

Only the bending stiffness of the beam is adjusted. This is an important distinction, because it is essential that the story drift and frame stiffness be estimated while the joint rotation is conservatively and at least approximately retained. The rotation of the column at the joint is the same for the deflections achieved with Equations C5-5 and C5-6. However, in Equation C5-5, the column rotation is achieved by the sum of a joint rotation, θ, and a beam end rotation. That is, the true joint rotation is somewhat smaller than the column rotation. Therefore, the rotation of the column at the joint is used conservatively as the joint rotation, θ, with this simplified analysis procedure.

While the spring stiffness of the connections must be considered in elastic analysis of PR frames, the elastic properties of the members are the same as those used in FR steel frames. Composite properties of the member should be used for encased members with the concrete encasement in contact with the steel. The stiffness of masonry infill walls, and other structural and nonstructural elements, should also be included as in the FR frame analysis.

Figure C5-5 shows a typical moment rotation hysteresis curve for a PR connection. The slope of this curve is the spring stiffness. For inelastic analysis, the computer models must recognize that the rotational spring stiffness of the connection changes dramatically with the deformation. These models are necessarily quite complicated, and relatively few computer models are available at this time. In nonlinear procedures, the variable rotational spring stiffness should be included in the computer model as illustrated in Figure C5-4. With this procedure, the rotational connections between the beams and columns is replaced by rotational springs with variable (nonlinear) spring stiffness. Direct transfer of shear and axial forces is permitted by the connection. A step-by-step nonlinear procedure can be performed by incremental changes in the rotational spring stiffness. The discussion provided in Section C5.4.3.3 on individual connection types provides insight into the variation of stiffness for different PR connections.

C5.4.3.3 Strength and Deformation Acceptance Criteria

The strength and deformation of PR frames are dominated by the connections. Member properties are identical to those used for members in FR frames, and are defined by plastic analysis techniques similar to those used by AISC (1994a). While composite action due to concrete encasement is seldom used in estimating the resistance of members in FR or PR frames, the engineer is encouraged to utilize both the stiffness and resistance provided by composite action for PR connections. This increased stiffness and resistance is particularly great for any of the weaker, more flexible connections.

The *m* factors used for the linear procedures and the deformation limits employed for nonlinear procedures are very sensitive to connection failure mode and the connection type. As a result, more detailed discussion of individual PR connection types is provided in this *Commentary*. The *m* factors and deformation limits are summarized in Tables 5-5 and 5-6. It should be emphasized that the limits for PR connections in these tables often require adjustment for deeper beams.

Flange Plate Connections. Flange plate connections that are welded to the column and bolted to the beam, as shown in Figure C5-7, are relatively stiff and strong PR connections. In fact, the flange plates could be designed for strength and stiffness such that the behavior could be classified as fully restrained (SAC, 1995). These connections exhibit fairly good hysteretic behavior with moderate pinching. Flange plate connections may also be welded to both the beam and the column as shown in Figure C5-8. Both types may be close to the stiffness limit required to qualify as an FR connection. They are relatively modern connections that are seldom encased in concrete for fire protection. Therefore, composite action due to encasement is not a major concern.

It is important that the failure modes considered in the analysis include plastic bending capacity of the beam, plastic capacity of the net section (including consideration of the critical row of bolts or the narrow point of a welded plate), resistance of the connectors (welds and bolts) themselves, local buckling of the flange plate, and weld strength between the flange plate and the column flange.

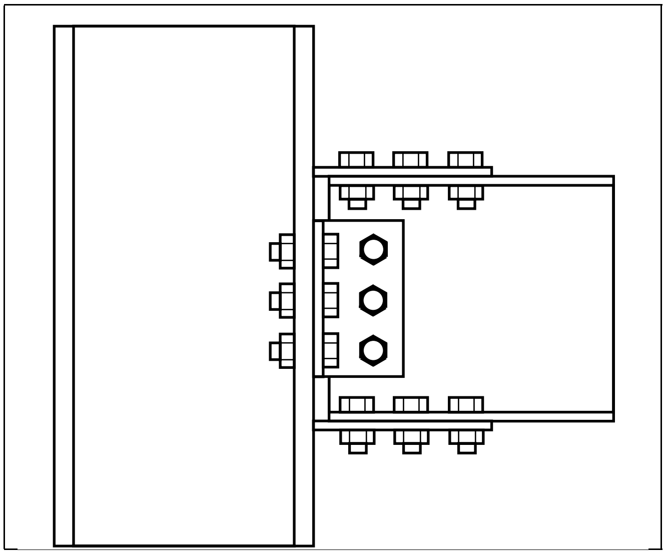


Figure C5-7 Bolted Flange Plate Connection

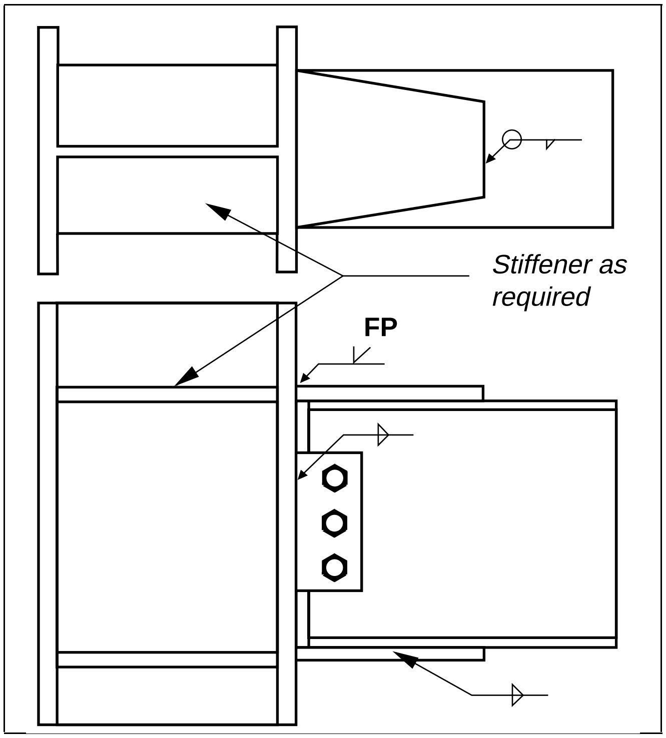


Figure C5-8 Welded Flange Plate Connection

The ductility appears to be greatest when the net section of the flange plate controls the resistance of the connection, and the ductility is lowest when weld resistance controls the strength of the connection. The moment capacity of the connection should be taken as the smallest moment produced by these different failure modes. The relative ductility of these different failure modes is considered in the definitions of *m* values and connection rotation limits in Tables 5-5 and 5-6. For more details on individual test results, see references by Popov and Pinkney (1969) and Harriott and Astaneh-Asl (1990).

End Plate Connections. End plate connections such as shown in Figure C5-9 are also stiff and strong PR connections, sometimes qualifying as an FR connection for stiffness analysis. Their use became more common around 1960, since they typically require high-strength bolts. This type of connection is most ductile if flexural yielding of the beam or the end plate occurs. It fails abruptly at small deformations if tensile failure of either the high-strength bolts or the weld occurs. These differences in relative ductility are reflected in the *m* values and deformation limits provided in Tables 5-5 and 5-6. It is important that the failure modes considered in the analysis include the plastic capacity of the beam, the local bending plastic capacity of the plate, the local bending plastic capacity of the column flange, the capacity of the fillet or penetration welds between the end of the beam and the end plate, and the tensile capacity of the bolts, including prying action.

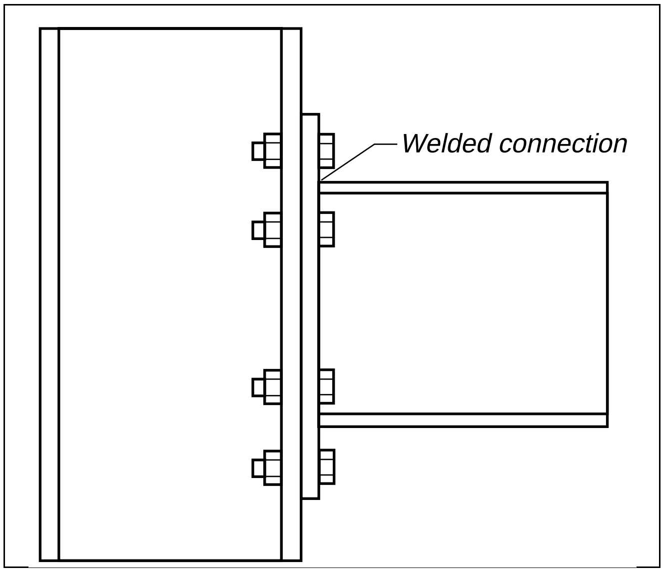


Figure C5-9 End Plate Connection

The moment capacity of the connection should be taken as the smallest moment produced by these different failure modes. However, it should be recognized that there is considerable uncertainty in the various calculations, and so the *m* values for thin plate failure modes (i.e., local bending of end plate) should be used only if the capacity achieved with all other failure modes exceeds the plastic bending of the end plate by 25%. The *m* value for thick plate or stiffened plate failure modes should be used only if the capacity achieved with all other failure modes exceeds the plastic bending capacity of the beam by 25%. Otherwise, the lower value should be employed. If these overstrength requirements are met, the AISC strength calculations appear to be appropriate for seismic evaluation.

Empirical models for connection nonlinear monotonic moment rotation behavior have been developed. The formula by Frye and Morris (1975) for end plates without column stiffeners is

$$\theta = 1.83 \times 10^{-3} \times (KM) + 1.04 \times 10^{-4} \times (KM)^3 + 6.38 \times 10^{-6} \times (KM)^5 \tag{C5-8}$$

where

$$K = d^{-2.4} \times t^{-0.4} \times f^{1.1} \tag{C5-9}$$

M = Applied moment

d = Distance between center of top and bottom bolt line

t = End plate thickness

f = Bolt diameter

θ = Rotation of end of beam relative to column

The formula by Frye and Morris (1975) for end plates with column stiffeners is

$$\theta = 1.79 \times 10^{-3} \times (KM) + 1.76 \times 10^{-4} \times (KM)^3 + 2.04 \times 10^{-4} \times (KM)^5 \tag{C5-10}$$

where

$$K = d^{-2.4} \times t^{-0.6}$$

More details on individual test results and failure modes for end plate connections are given in Tsai and Popov (1990), Johnstone and Walpole (1981), Whittaker and Walpole (1982), Murray and Kukreti (1988), and Sherbourne (1961).

T-Stub Connections. T-stub connections have been used for at least 70 years; Figure C5-10 illustrates a typical connection. Riveted details such as those illustrated in the figure were used for the first half of this period; high-strength bolts have been used in more recent practice. During the early part of this period, these connections were encased in massive, lightly reinforced concrete for fire protection. T-stub connections are of intermediate strength and stiffness, but approach FR behavior if carefully designed. The connection will seldom develop the full plastic capacity of the beam, but it will develop a significant portion of this beam-bending capacity. As a result, composite action due to the concrete encasement will often significantly increase the strength and rotational spring stiffness of the connection.

A number of failure modes are possible with these connections. The *m* values and deformation limits are very sensitive to the failure mode. Greater ductility and larger inelastic deformations can be achieved in connections with flexural yielding in the flanges of the T-sections. The smallest ductility and inelastic deformation can be achieved on connections where the inelastic deformation is controlled by the tensile connectors between the T-section and the column flange. The limits established in Tables 5-5 and 5-6 reflect these differences in behavior. The guidelines provided in Section 5.4.3.3 provide approximate estimates of the resistance and failure mode of T-stub connections. Accurate calculation of the connection failure modes and resistance is difficult because of the interaction between flexure in the flanges and tension in the connectors through prying action in the connection. As a result, the equations in the *Guidelines* are very approximate and quite conservative in their estimates of the resistance.

More detailed procedures have been developed for estimation of the connection resistance and failure mode. These procedures are considerably more accurate, but they require more effort and calculation. They also permit consideration of composite action due to concrete encasement. One such procedure for riveted T-stub connections is outlined below in this *Commentary.*

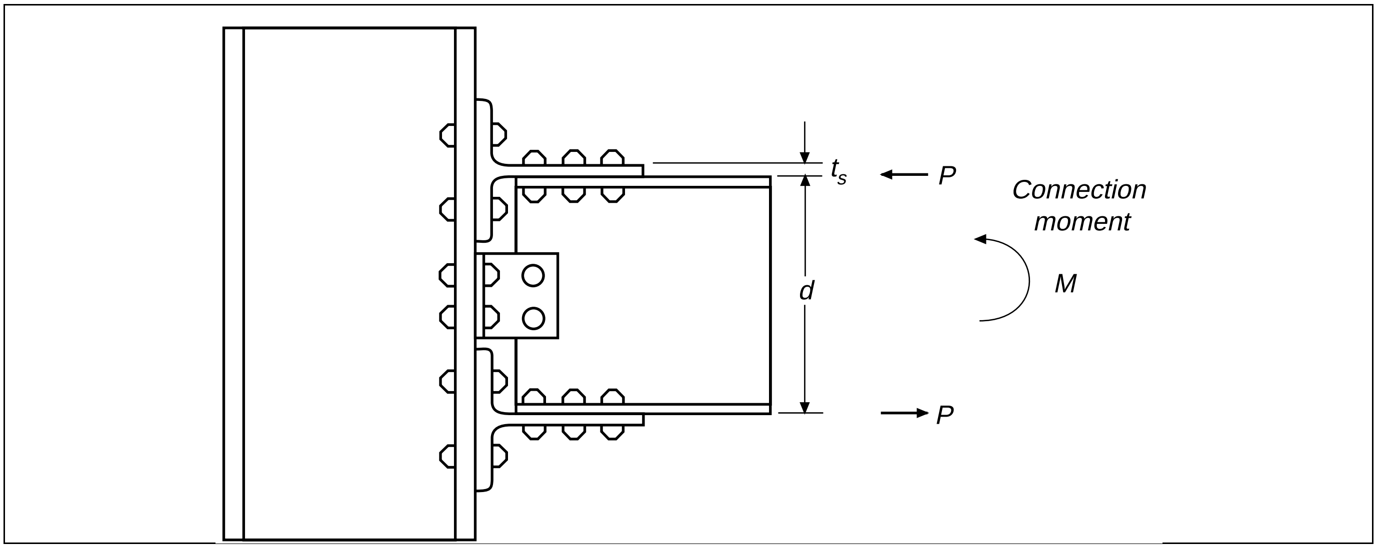


Figure C5-10 T-Stub Connection

For riveted bare steel connections, Figure C5-10 illustrates the general configuration of the connection. The connection moment can be approximated with the flange forces, P, as shown in the figure. The maximum flange force can be determined by examining a number of different failure modes and determining which mode leads to the smallest flange force. The flange force can then be directly translated into a moment capacity, M_{CE}, of a bare steel connection, or it can be combined with other calculations to predict M_{CE} for an encased connection.

T-Stub Connections: Plastic Moment Capacity of the Beam. The ultimate capacity of the connection is limited by the expected plastic capacity of the beam, so that $M_{CE} < Z F_{ye}$, where Z is the plastic section modulus of the steel and F_{ye} is the expected yield stress of the beam.

T-Stub Connections: Shearing of Rivets Between the Beam Flange and the T-Section. The expected force, P_{CE}, must be transferred from the beam flange to the stem of the T-section. The shear strength of the connectors provides another limit on the moment capacity, so that

$$P_{CE} \le A_c F_{ve} N_{Stem} \tag{C5-11}$$

and

$$M_{CE} = P_{CE} d_b \tag{C5-12}$$

where

d_b = Beam depth

A_c = Gross cross-sectional area of a single connector

F_{ve} = Expected shear strength of the connector

N_{Stem} = Number of connector shear planes

T-Stub Connections: Tension in the Stem of the T-Section. The ultimate tensile capacity of the stem (or web) of the T-section may also control the resistance of the connection, and it should be checked by the normal AISC tension member criteria; that is,

$$P_{CE} \le F_{ye} A_g \tag{C5-13}$$

$$P_{CE} \le F_{te} A_e \tag{C5-14}$$

and

$$M_{CE} \le P_{CE}(d + t_s) \tag{C5-15}$$

where

F_{ye} = Expected yield of steel in T-section stem
F_{te} = Expected tensile strength of steel in T-section stem
A_e = Net effective area of stem
A_g = Gross area of stem
t_s = Thickness of stem

T-Stub Connections: Local Plastic Bending of Flange of T-Section. Flexure of the flange of the T-section must also be considered. Prying forces are necessary to develop these flexural moments, and the prying forces increase the tensile forces in the connectors. Prying action plays a different role in older steel connections than it does in connections with modern high-strength bolts. Mild steel rivets yield and elongate more in tension than do high-strength bolts. This tensile yielding limits the prying action, so that a balance between flexure and tensile yield may occur. Flexure of the flange has the equilibrium conditions described in Figure C5-11. The local flange moments are limited by the plastic bending capacity of the flange, and this limits the force, P_{CE}. Thus, the ultimate capacity of the T-stub connection is approximated by

$$P_{CE} \le \frac{w t_s^2 F_{ye}}{d'} \qquad \text{(C5-16)}$$

and

$$M_{CE} < P_{CE}(d + t_s) \qquad \text{(C5-17)}$$

where d' is as shown in Figure C5-11 and t_s is the thickness of the stem.

Equations C5-16 and C5-17 limit the capacity of the connection based on flexure in the connecting elements. However, this flexure requires a prying force, as can be seen in Figure C5-11. The prying force introduces an additional tension in the tensile connectors, and a coupled mode of failure may occur. As a result, the capacity may be reduced to

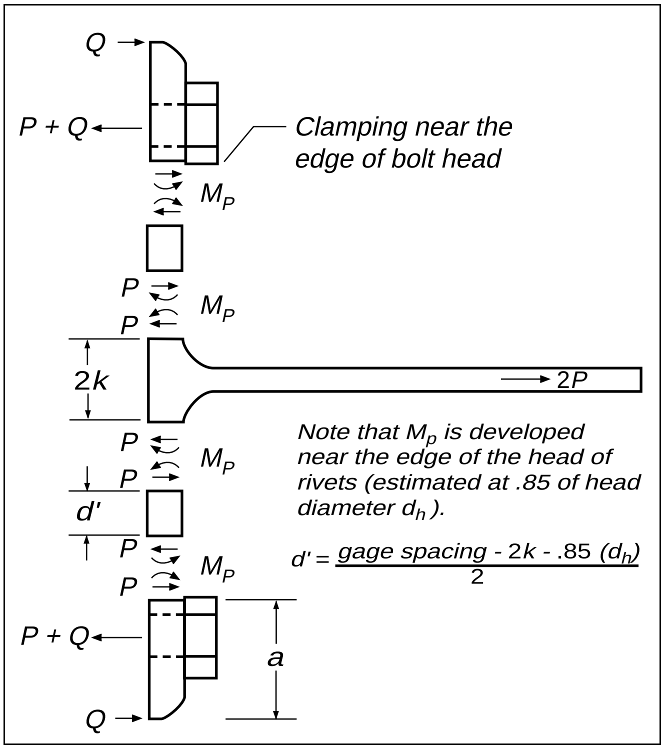


Figure C5-11 Prying Action in T-Stub Connection

$$P_{CE} \le \frac{\dfrac{0.5 w t_s^2 F_{ye}}{a} + (F_{ye} A_c N_{VL})}{1 + \dfrac{d'}{a}} \qquad \text{(C5-18)}$$

$$M_{CE} \le (d + t_s/2) P_{CE} \qquad \text{(C5-19)}$$

N_{VL} is the number of tensile connectors between the flange of the T-section and the column flange.

T-Stub Connections: Tension of Rivets Between T-Section and Column. The tensile capacity of the connectors between the vertical leg of the angle or T-section and the column face may also control the resistance of the connection.

The equations

$$P_{CE} = F_{ye} A_c N_{VL} \qquad \text{(C5-20)}$$

and

$$M_{CE} \le (d + t_s/2) P_{CE} \qquad \text{(C5-21)}$$

can be used for the T-stub connection.

N_{VL} = Number of connectors acting in tension

A_c = Net area of each connector

t_s = Thickness of the T-stub stem

d = Vertical distance to the center of the connectors

F_{ye} = Expected yield stress of the connectors

These equations limit the moment capacity of the connection based on the tensile capacity of the connector. If the above equations produce the smallest moment capacity of the connection, the connection capacity may be further reduced by

$$P_{CE} \leq \frac{0.5 w t_f^2 F_{ye}}{d'} \tag{C5-22}$$

and

$$M_{CE} \leq (d + t_s/2) P_{CE} \tag{C5-23}$$

where

w = Length of T-stub, in.

t_f = Thickness of T-stub flange, in.

for a T-stub connection if Equation C5-22 or C5-23 produces a smaller moment capacity than Equation C5-20 or C5-21, respectively.

Web connectors and composite action due to encasement for fire protection may contribute to the resistance of these connections. The later commentary on clip angle connection design methods will describe methods for incorporating these added factors. However, it should be noted that the additional capacity provided by the web connection and composite action due to concrete encasement is likely to be relatively small for T-stub connections, because the flange connection is relatively strong.

The resistance predicted by the previous procedure will usually be larger than that predicted by Equations 5-23 and 5-24, and the stiffness can be estimated by combining this resistance with Equations C5-2 and C5-3. The stiffness of bare steel connections can also be estimated by application of a secant modulus to empirical equations such as

$$\theta = 2.1 \times 10^{-4} \times (KM) + 6.2 \times 10^{-6} \times (KM)^3 - 7.6 \times 10^{-9} \times (KM)^5 \tag{C5-24}$$

where

$$K = d^{-1.5} \times t^{-0.5} \times f^{1.1} \times L^{-0.7} \tag{C5-25}$$

M = Connection moment, kip-in.

d = Depth of beam, in.

t = Thickness of clip angle plus column flange, in.

f = Bolt diameter, in.

L = Length of T-stub section, in.

θ = Rotation of end of beam relative to column, rad

More information on individual test results and failure modes for T-stub connections is given by Roeder, Leon, and Preece (1994), Hechtman and Johnston (1947), Rathbun (1936), and Batho and Lash (1936).

Clip Angle Connections. Clip angle connections, as illustrated in Figure C5-12, have a similar history to that of T-stub connections. Rivets were used until about 1960, and high-strength bolts have been used more recently. For many years, the connections were encased in massive, lightly reinforced concrete for fire protection. Clip angle connections are among the weaker and more flexible PR connections. The connection will usually develop only a small portion of the plastic capacity of the beam. As a result, composite action due to the concrete encasement will at most invariably provide a significant increase to the strength and rotational spring stiffness of the connection. A number of failure modes are possible with clip angle connections. The *m* values and deformation limits provided in Tables 5-5 and 5-6 are based on the failure mode. Greater ductility and larger inelastic deformations can be achieved in connections with flexural yielding in the outstanding leg (OSL) of the clip angle. The smallest ductility and inelastic deformation occurs when the resistance is controlled by the tensile connectors between the OSL and the column flange. The limits established in Tables 5-5 and 5-6 reflect these priorities. The prediction of the failure mode of these connections is very important. Equations 5-17 through 5-22 of Section 5.4.3.3 of the

Guidelines provide approximate equations for estimating the resistance and failure mode. Accurate calculation of the connection failure modes and resistance is difficult because of the interaction between flexure in the flanges and tension in the connectors through prying action in the connection. As a result, the equations in the *Guidelines* are very approximate and conservative.

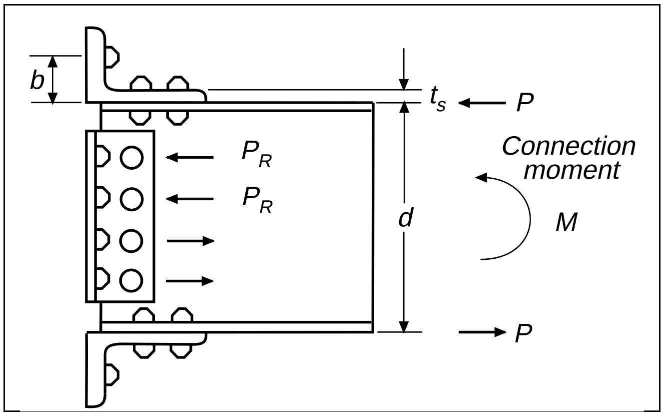


Figure C5-12 Clip Angle Connection

More detailed procedures have been developed for estimation of the connection resistance and failure mode. These procedures are more accurate, but they require more effort and calculation. They also permit consideration of composite action due to concrete encasement. One such procedure for riveted clip angle connections is outlined in this *Commentary*, as follows.

For riveted bare steel clip angle connections, Figure C5-12 illustrates the general configuration of the connection. The connection moment can be approximated with the flange force, *P*. The expected flange force, P_{CE}, can be determined by finding the smallest force provided by different failure modes. The flange force can then be directly translated into a moment capacity, M_{CE}, of a bare steel connection, or it can be combined with other calculations to predict M_{CE} for an encased connection.

Clip Angle Connections: Shearing of Rivets Between the Beam Flange and the Clip Angle. The force, *P*, must be transferred from the beam flange to the OSL of the clip angle. The shear strength of the connectors provide one limit on the moment capacity, so that

$$P_{CE} \le A_b F_{Ve} N_{OSL} \tag{C5-26}$$

and

$$M_{CE} = P_{CE} d_b \tag{C5-27}$$

where

d_b = Beam depth
A_b = Cross-sectional area of single connector
F_{Ve} = Expected shear strength of connector
N_{OSL} = Number of connector shear planes in OSL of angle

Clip Angle Connections: Tension of Outstanding Leg (OSL) of Clip Angle. The ultimate tensile capacity of the OSL may also control the resistance of the connection, and it should be checked by the normal AISC tension member criteria; that is,

$$P_{CE} \le F_{ye} A_g \tag{C5-28}$$

$$P_{CE} \le F_{te} A_e \tag{C5-29}$$

and

$$M_{CE} \le P_{CE}(d_b + t_s) \tag{C5-30}$$

Clip Angle Connections: Local Plastic Bending of Flange of Clip Angle. Flexure of the vertical leg of the angle must also be considered. Prying forces are necessary to develop these flexural moments, and the prying forces increase the tensile forces in the connectors. However, prying action plays a different role in older riveted connections than it does in connections with modern high-strength bolts. Mild steel rivets yield and elongate more than high-strength bolts and this limits the prying action. In a clip angle connection, the flexure of the vertical flange has the equilibrium conditions described in Figure C5-13. The moments *M2* and *M4* limit the force, *P*, that can be transferred by the vertical leg. They are also limited by the plastic bending capacity of the leg. Thus,

$$P_{CE} \le \frac{0.5 w t_s^2 F_{ye}}{d' - \frac{t_s}{2}} \tag{C5-31}$$

and

$$M_{CE} < P_{CE}(d + d') - \frac{0.25 w t_s^2 F_{ye}}{d' - \frac{t_s}{2}} \qquad \text{(C5-32)}$$

where d' is as defined in the figure and w is the length of the angle.

Clip Angle Connections: Prying Forces and Tension of Rivets Between Clip Angle and Column. Flexure requires a prying force, as can be seen in Figure C5-14. The prying force introduces an additional tension in the tensile connectors, and a coupled mode of failure may occur. As a result, the capacity of the connection produced by Equation C5-32 may be reduced by

$$P_{CE} \le \frac{\frac{0.25 w t_s^2 F_{ye}}{a} + (F_{ye} A_c N_{VL})}{1 + \frac{d' - \frac{t_s}{3}}{a}} \qquad \text{(C5-33)}$$

$$M_{CE} \le P_{CE}(d + d') - (F_{ye} A_c N_{VL} - P_{CE})a \qquad \text{(C5-34)}$$

N_{VL} is the number of tensile connectors between the angle and the column flange. The prying force may be relieved, however, by tensile yielding of the connector. Under these conditions, the tensile capacity of the connectors between the vertical leg of the angle and the column face may directly control the resistance of the connection; that is,

$$P_{CE} = F_{ye} A_c N_{VL} \qquad \text{(C5-35)}$$

and

$$M_{CE} \le (d + b) P_{CE} \qquad \text{(C5-36)}$$

where b is the vertical distance to the center of the connectors as shown in Figure C5-14, and F_{ye} is the expected yield strength of the connectors.

Web connectors and composite action due to encasement for fire protection may contribute to the resistance of these PR connections. These contributions may be particularly significant for the clip angle connections, because the clip angle flange connection is weaker than most other PR connections. The procedures for calculating these additional contributions are similar for all types of PR connections, and a brief description of procedures for completing this calculation follows.

Contribution of Web Connection to Moment Capacity. The smallest moment capacity, M_{CE}, and its associated flange force, P_{CE}, obtained in previous calculations, determine the mode of failure and moment capacity of the bare steel flange connection. The web connection also contributes to the moment capacity as illustrated in Figure C5-12. The web connectors develop forces that combine to form couples as illustrated in the figure. The calculations required to determine the forces developed in the web are similar to those used in determining the moment capacity provided by the flange connection. The addition of the web connector moment generally improves the estimate of the ultimate capacity of the connection, since past research has indicated that consideration of only the moment capacity contributed by the flanges will underestimate the true resistance. The underestimate is particularly significant for weaker and more flexible PR connections such as clip angle connections. However, a larger rotation is required to develop this additional moment in the web connection than is required to develop the moment capacity of the flanges. Thus, some connections with limited rotational ability—such as those with tensile failure of the column flange connectors—will not be able to develop this additional moment capacity. The additional moment capacity due to the web connection can be added to the contribution of the flange connection.

Contribution of Composite Action to the Moment Capacity. For encased connections, composite action develops additional moment resistance that can be considered. The critical mode of failure for the flange connections of the bare steel is again determined by the procedures described earlier for determination of the moment capacity due to the flange bare steel connection. For this mode of failure, the critical tensile flange force, P_{CE}, and the centroid of the location of this tensile force remain unchanged after the connection is encased. This tensile force is then balanced by the compressive force of the concrete using the normal ACI Ultimate Strength Design rectangular stress block, as illustrated in Figure C5-14. The location of the neutral axis and the ultimate capacity are readily determined by equilibrium calculations. These calculations again

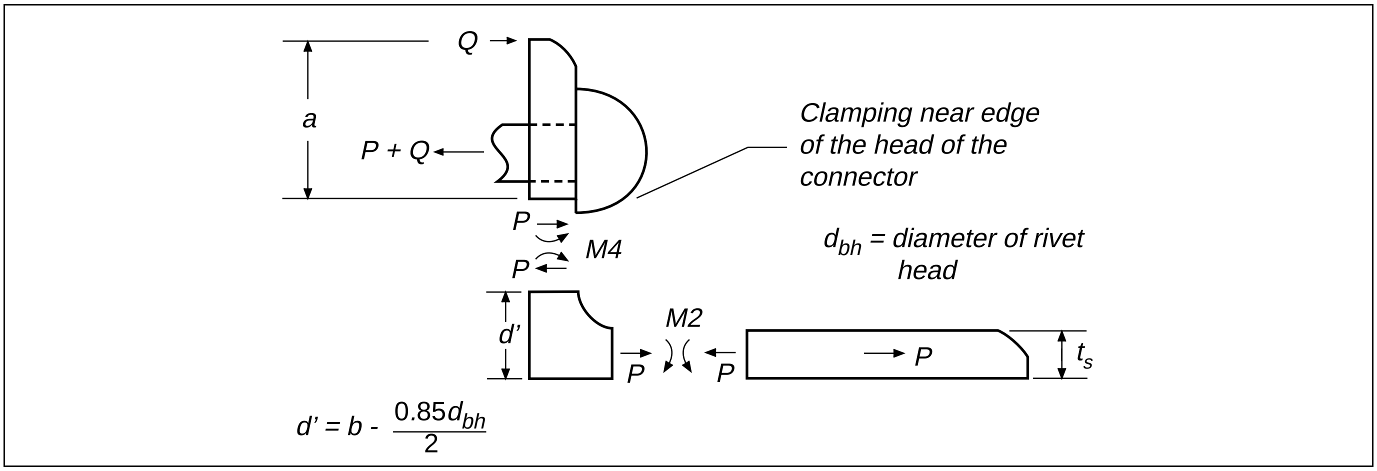


Figure C5-13 Forces in Clip Angle

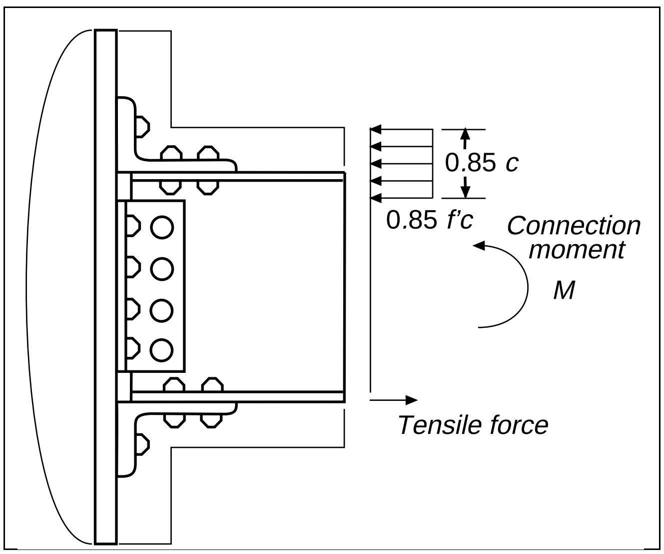


Figure C5-14 Moment Resistance by Clip Angle Connection

neglect the capacity of the web connectors, and past research has shown this to be a lower bound of the connection resistance.

The web connectors should also be considered, as illustrated in Figure C5-15. The web connectors are primarily in tension when the connection is encased, as illustrated in the figure. Flange connectors for the compression flange may be included if they are located well above the neutral axis. The tensile capacity of the web connectors is included only if they are located well below the neutral axis. A larger rotation is required to activate the web connectors in composite action than is required to activate the moment resistance of the flange connectors. Connections that developed a large rotation, such as those with flexural yielding of the clip angle, easily develop the moment resistance predicted by the model with composite actions. Some connections with smaller rotational capacity, such as those with tensile yield of connectors, do not develop the full composite moment resistance, including the web connection. The calculated moment capacity with web connectors and composite action may be larger than the experimental values in a few cases. However, this prediction is consistently closer to the true moment capacity of the connection. The moment capacity calculated by this procedure is all-inclusive, and it should not be added to the bare steel contributions.

The resistance predicted by the previous procedure will usually be larger than that predicted by Equations 5-17 and 5-22, and the stiffness can be estimated by combining this resistance with Equations 5-14 and 5-15. The stiffness of bare steel connections can also be estimated by application of a secant modulus to empirical equations such as

$$\theta = 0.2232 \times 10^{-4} \times (KM) + 0.1851 \times 10^{-7} \times (KM)^3 - 0.3289 \times 10^{-11} \times (KM)^5 \qquad \text{(C5-37)}$$

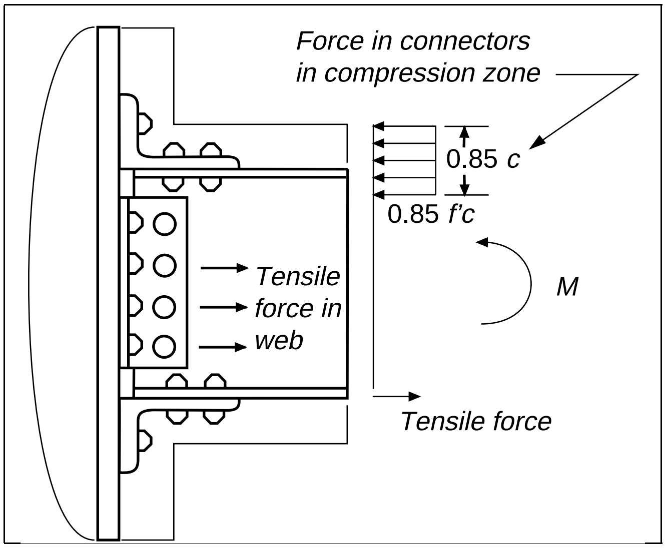


Figure C5-15 Effects of Web Rivets and Slab

where

$$K = d_b^{-1.2870} \times t^{-1.1281} \times t_a^{-0.6941} \times L^{-0.6941} \times \left(g - \frac{f}{2}\right)1.3499 \quad \text{(C5-38)}$$

g = Gage in flange angle
t = Thickness of clip angle
t_a = Thickness of web angles
f = Bolt diameter
L = Length of clip angles
M = Connection moment
θ = Rotation of end of beam relative to column

More information on individual test results and failure modes for T-stub connections may be found in Roeder et al. (1994), Azizinamini and Readziminski (1989), Hechtman and Johnston (1947), Rathbun (1936), Batho and Lash (1936), and Batho (1938).

C5.4.3.4 Rehabilitation Measures for PR Moment Frames

As stated in the *Guidelines*, many of the rehabilitation measures given for FR frames also apply to PR frames (see Section 5.4.2.4).

Older PR moment frames may be too flexible even if the beams and columns are encased in concrete. If this is the case, additional stiffness may be achieved by several means. Steel braces may be added in either a concentric or eccentric manner. Reinforced concrete or masonry infills may be added to some of the bays of the frames. Methods for designing and/or evaluating the effects of infills are given in the *Guidelines* Chapters 6 and 7. New steel frames may be attached to the outside of the building, but connections and load paths must be checked carefully.

C5.5 Steel Braced Frames

C5.5.1 General

Braced frames do not appear to be too common in seismic areas before the 1950s and 1960s, even though their use has been dated back to the 1920s in nonseismic areas. In the earlier applications, the bracings appear to have played a secondary role in lateral-load-carrying function, with the primary frames being moment frames and masonry infilled frames. They have generally taken the form of light vertical trusses, which were often knee-braced types. These older vertical trusses were connected by rivets and generally encased in fireproofing concrete, and they generally did not develop the capacity of the members. Use of bracings has, however, been common in one- and two-story structures, especially industrial types. Tension-only diagonals have often been used in one- and two-story applications.

More complete braced-frame systems started evolving after the 1950s, especially in low- to nonseismic areas. Braced frames were still generally combined with moment frames as dual frames in seismic areas.

Diagonal members and their connections form the basic components. The brace member may consist of single or double angles, channels or T-sections, circular or rectangular tubes with or without concrete filler, or tension rods or angles. The connection of the brace to the frame is generally by gusset plates with rivets, bolts, or welding.

C5.5.2 Concentric Braced Frames (CBFs)

C5.5.2.1 General

Concentric braced frames (CBFs) are very efficient structural systems in steel for resisting lateral forces due to wind or earthquakes because they provide complete truss action. That is the main reason for their popularity.

However, this framing system has not been considered as ductile in past or current design practice for earthquake resistance. The nonductile behavior of these structures mainly results from early cracking and fracture of bracing members or connections during large cyclic deformations in the post-buckling range. The reason lies in the code philosophy. Instead of requiring the bracing members and their connections to withstand cyclic post-buckling deformations without premature failures (i.e., for adequate ductility), the codes generally specify increased lateral design forces. It has recently been recognized that CBFs designed according to the past or current code procedures may not survive a major earthquake without serious consequences.

During a severe earthquake, bracing members in CBFs experience large deformations in cyclic tension in the post-buckling range, which cause reversed cyclic rotations to occur at plastic hinges in much the same way as they do in beams and columns of moment frames. In fact, braces in a typical CBF should be expected to yield and buckle at rather moderate story drifts of about 0.3% to 0.5%. In a severe earthquake the braces could undergo post-buckling axial deformations up to 10 to 20 times their tension yield deformation. In order to survive such large cyclic deformations without premature failure, the bracing members and their connections must be properly detailed. This often has not been the case in past design practice.

Early brace failures were observed in testing of the United States-Japan full-size, six-story structure with hollow tubular bracing in an inverted V pattern (Foutch, Goel, and Roeder, 1987). Two recently completed analytical studies (Tang and Goel, 1987; Hassan and Goel, 1991) investigated the seismic behavior due to severe ground motions of a number of concentric-braced structures designed according to different design philosophies. Included in the studies were CBFs with and without backup moment frames. It was found that structures designed strictly in accordance with the 1988 UBC procedure showed early brace fractures leading to large story drifts of up to 6% to 7% or more, which results in excessive ductility demands on beams and columns.

In the post-buckling range of a bracing member, local buckling of compression elements limits the plastic moment capacity and, consequently, the compression load capacity of the member. More importantly, however, the extent and severity of local buckling has a major influence on fracture life (ductility) because of high concentration of reversed cyclic strains at those locations. Therefore, in order to prevent early fracture of bracing members, their width-thickness ratios (compactness) must be kept within much smaller limits than those used in current practice. For rectangular tubular sections, a limit of $95/\sqrt{F_y}$ has been suggested (Tang and Goel, 1987), which is half of that specified in AISC (1994a). This is reasonable because plastic design is based on ductility under monotonic loading, whereas seismic design counts on the ability of structural elements to withstand large cyclic inelastic deformations in the event of a severe earthquake.

If the ductility of bracing members is ensured by using compact sections, as suggested above, and other frame members are properly designed by considering the strength of the braces, there is no need to use increased seismic design forces for a CBF. Thus, a number of structures were designed by using compact rectangular tubular bracing members $(b/t < 95/\sqrt{F_y})$ and $R_w = 12$ (same as specified by the 1988 UBC for SMRF). Also, the "penalty factor" of 1.5 (1988 UBC) was deleted in calculating the forces in chevron braces. Dual systems, as well as those without backup Special Moment Frames, were designed by this approach, and their responses to several severe ground motion records (peak accelerations of about 0.5g) were studied. No brace fractures occurred in these frames and their responses were much better than those of the code-designed structures. The story drifts were generally under 3%. The hysteretic loops of shear force in the first story of a ductile braced structure with backup SMRF are shown in Figure C5-16.

As mentioned earlier, local buckling has been found to be the most dominant factor influencing the ductility and energy dissipation capacity of bracing members. For rectangular tube sections, which are very popular for braces, an alternative to using smaller width-thickness ratios is to use plain concrete infill. Concrete infilling has been found to reduce the effective width-thickness ratio by as much as 50%, thus increasing the fracture life by up to 300% (Lee and Goel, 1987). The width-thickness ratio of angle sections should be kept under $52/\sqrt{F_y}$. Double angles used in toe-to-toe shape perform much better than the conventional back-to-back configuration (Aslani and Goel, 1989). For built-up sections, such as double angles or double channels, a stitch spacing such that L/r of the individual elements

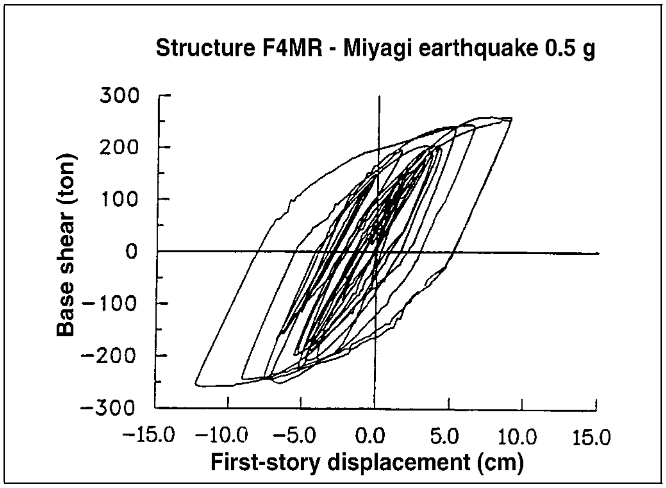


Figure C5-16 Response of Braced Story with Moment Frame Backup

does not exceed 0.4 times the *KL/r* of the overall member was recommended (Xu and Goel, 1990). For single gusset plate connections in members buckling out of plane, the gusset plates should have a clear length of about two times their thickness in order to allow for restraint-free plastic rotations during cyclic post-buckling of the member (Astaneh-Asl et al., 1986). Some of these recommendations, such as using concrete infill in tubular members and increasing the number of stitches in built-up members, can be used in seismic upgrading of existing structures.

As a result of the research findings discussed above, provisions were introduced for Special (ductile) Concentric Braced Frames (SCBF) in the 1994 *Uniform Building Code* and the 1994 NEHRP *Recommended Provisions for Seismic Regulations for New Buildings* (BSSC, 1995). The older provisions for CBFs were retained as applicable to Ordinary Concentric Braced Frames (OCBF). In both provisions the R_w or R factors were adjusted to reflect the additional requirements to ensure ductile behavior of bracing members.

C5.5.2.2 Stiffness for Analysis

The purpose of a Linear Static or Dynamic Procedure is to evaluate the acceptability of components, elements, and connections in a rather simplistic manner. Unlike other framing systems, seismic behavior and performance of a CBF are very much governed by those of the bracing members and their connections. Use of a linear procedure for evaluation purposes is usually based on the premise that the component is capable of reaching maximum displacements under expected reversed cyclic deformations without any major drop in actual strength. Since this is usually not the case for a CBF, the factor C_3 is introduced in Section 3.3.1. Also, the *m* values as given in Table 5-7 have been derived by taking the pertinent factors into consideration. Professional judgment should be applied as appropriate. Use of Nonlinear Static or Dynamic Procedures is highly recommended for more precise evaluation.

The major components of a CBF are beams, columns, and braces. Because of the truss action, a CBF is considerably stiffer than a moment-resisting frame of equal strength, prior to buckling or yielding of bracing members at moderate story drift levels. Under increasing story drifts, the buckling of compression braces is followed by yielding of tension braces, after which the truss action partially breaks down, but the columns develop very substantial additional shear strengths through flexure. The strength and stiffness contribution of columns comes not only from those in the braced bays, but also from all other columns that are designed to support gravity loads only. This is because the columns in steel frames are generally made continuous even when the beam-to-column connections are not moment-resisting. Thus, CBF structures can possess very substantial overstrength after buckling of the compression braces. For nonlinear procedures, all columns may be included in the model with proper regard to their continuity and base connection details.

The force-deformation behavior of a brace is governed by the tension yield force, $P_y = AF_y$, the compression buckling load, and the post-buckling residual compression force, which are functions of the yield stress and the slenderness ratio of the brace. The residual force is also influenced by compactness, cross-section shape, and other details of the member. A typical force versus axial deformation response of a steel brace is shown in Figure C5-17. For this brace the residual force was about 20% of the buckling load, a percentage that is about the same for many brace configurations. Tests on a variety of bracing members have been carried out at the University of Michigan (Gugerli and Goel, 1982; Aslani and Goel, 1989). Other test results for brace components are available from the following sources: Lee and Goel, 1987; Xu and Goel, 1990; Fukuta et al., 1989; Goel and El–Tayem, 1986; Fitzgerald et al., 1989; Astaneh-Asl et al., 1986. Results of testing and/or analysis of braced frame elements have been reported by the following: Khatib et al., 1988; Ricles and Popov, 1987; Khatib et al., 1987; Bertero et

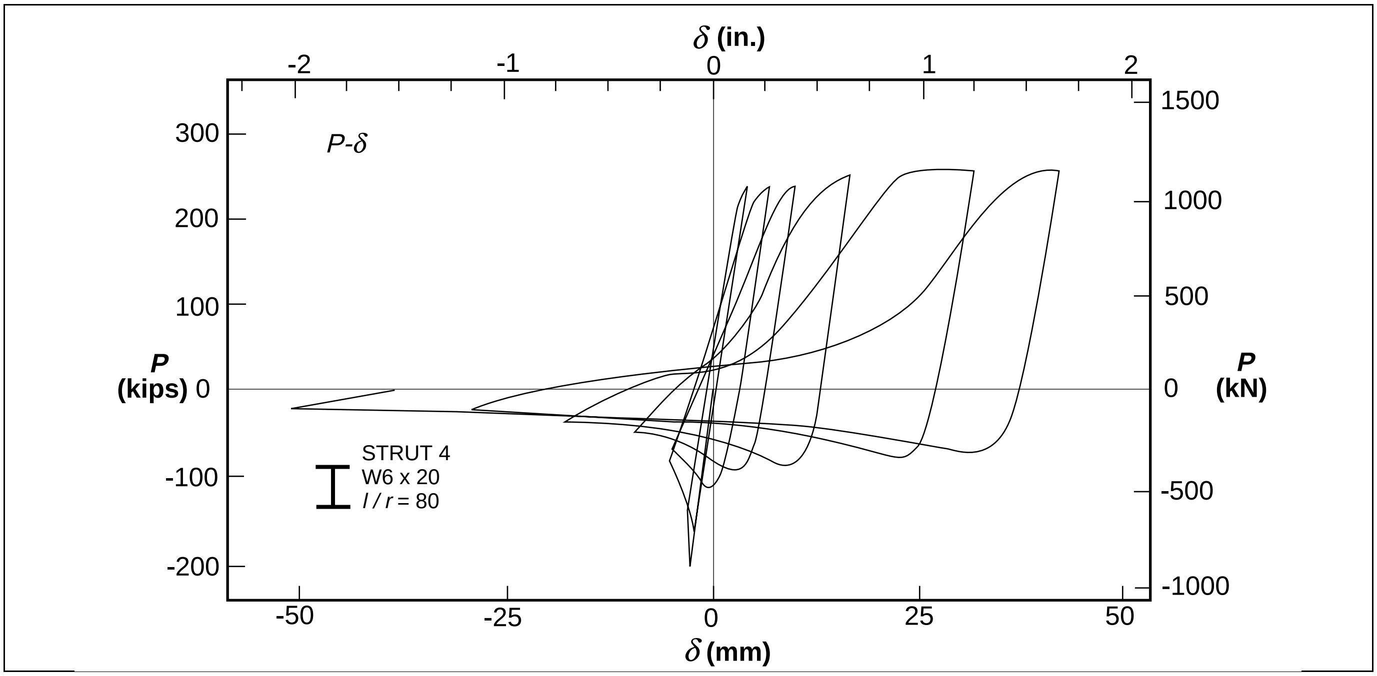


Figure C5-17 Typical Load versus Axial Deformation Behavior for a Brace

al., 1989; Wijanto et al., 1992; Uang and Bertero, 1986; Takanashi and Ohi, 1984; Midorikawa et al., 1989; Whittaker et al., 1989; Goel, 1986; Redwood, et al., 1991; Wijanto et al., 1992; Foutch et al., 1987; Roeder, 1989; Fukuta et al., 1984; Bertero et al., 1989; and Yang, 1984.

The hysteretic behavior of a brace may be modeled fairly accurately by using phenomenological models (Jain and Goel, 1978) or physical theory models (Ikeda and Mahin, 1984). The axial force versus axial deformation behavior of the Jain-Goel model is shown in Figure C5-18. A brace model similar to this should be used for Nonlinear Static or Dynamic Procedures (Rai, Goel, and Firmansjah, 1995). For a more simplified NSP, the axial force-deformation behavior of a brace in compression could be modeled as an elasto-plastic element with the yield force equal to the residual force. The residual force can be determined from Table 5-8 and Figure 5-1. However, an elastic analysis would also need to be done to determine the maximum axial force delivered to the column, the beam, and the beam-column connections.

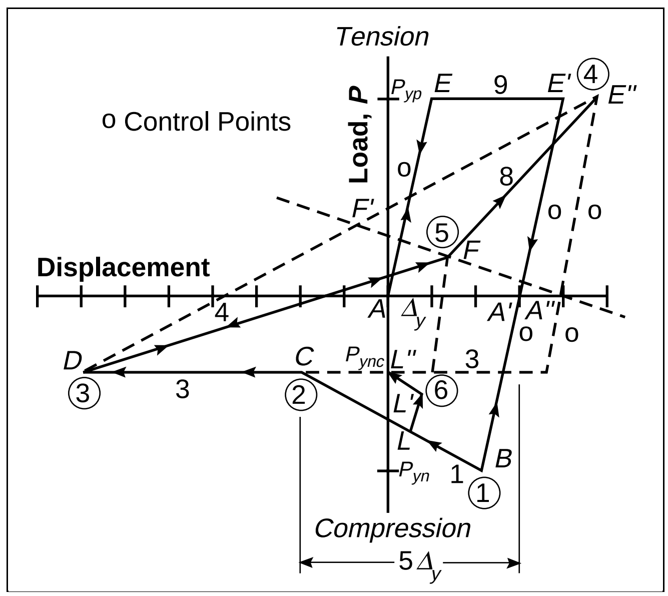


Figure C5-18 Axial Hysteresis Model—Load Starting in Tension

C5.5.2.3 Strength and Deformation Acceptance Criteria

The effective length factor is very important for calculating the expected strength of the brace. For diagonal, V, or inverted V braces attached to the column and beam with gusset plates through welded connections, the clear length of the brace should be used with a *k* of 0.8 for in-plane buckling and 1.0 for out-of-plane buckling. For bolted connections, a *k* value of 0.9 should be used.

C5.5.2.4 Rehabilitation Measures for Concentric Braced Frames

A. Component Strength Enhancement

Columns. The provisions for rehabilitating columns in moment frames are applicable to CBFs.

Beams. Provisions are the same as for moment frames:

Braces. Rehabilitation measures for braces include the following:

- Shear—Add steel plates parallel to the shear force, or encase in concrete.
- Moment—Add steel plates or encase in concrete.
- Axial —Add steel plates to increase section strength and/or reduce member slenderness; encase in concrete; provide secondary bracing members to reduce unbraced length; or replace with a section with greater capacity.
- Combined stresses—Use measures similar to those for axial braces.
- Stability—Stiffen element or connections by additional steel plates; provide secondary bracing elements; encase in concrete; or replace with a section with greater capacity.
- Concrete encasement—Remove or modify in cases where concrete causes undesirable failure mode.
- Element section properties
  - High b/t ratios—Infill with concrete, or replace with different section.
  - Spacing or capacity of stitch plates—Strengthen existing stitch connections, or provide stitch plates. If stitch plates are already in place, provide additional stitch plates.

Connections. Rehabilitation measures for connections include the following.

- Brace connections—Add welds or bolts; replace rivets with high-strength bolts; add plates to strengthen the connection.
- Concrete encasement—Remove or modify in cases where concrete causes an undesirable failure mode.
- Column base strength—Use same measures as for moment frames.

System Enhancements. The following system enhancements should be considered:

- “K” bracing—Remove bracing or strengthen column such that strength and stiffness are sufficient to transfer maximum bracing forces.
- Knee bracing—Use the same measures as for “K” bracing.
- Chevron bracing—Strengthen beam as required to develop maximum unbalanced bracing loads.
- Tension-only systems—Replace bracing with elements capable of resisting compression loads, or add stiffening elements.

B. Rehabilitation Measures for Deformation Deficiencies

The following rehabilitation measures for adding stiffness to the building should be considered.

- Add steel plates.
- Encase in concrete.
- Replace existing braces.
- Add concrete or masonry infills.
- Add reinforced concrete shear walls.

C5.5.3 Eccentric Braced Frames (EBF)

C5.5.3.1 General

The eccentrically braced frame represents a hybrid framing system that is both stiff and ductile. The presence of the link beam, created by offsetting the point of action of the braces that frame into a beam, is primarily responsible for both the high stiffness of the frame and the good ductility characteristics.

The link beam is called short if $e < 1.6M_p/V_n$, and long if $e > 2.6M_p/V_n$, where e is the length of the link, M_p is the nominal plastic moment capacity of the section, and V_n is the nominal plastic shear capacity of the section. Links in the intermediate range of lengths are subject to interaction between moment and shear. A short link is stiffer than a long link, but it is also prone to greater ductility demands. Frame stiffness decreases rather rapidly with link length. The length of a link is generally chosen to maximize frame stiffness within the limits of available link ductility.

C5.5.3.2 Stiffness for Analysis

Elastic shearing deformations are important to the stiffness of the link element, which is typically modeled as a beam. The stiffness associated with flexural deformation is given by

$$K_b = \frac{12EI}{e^3} \tag{C5-39}$$

where E is Young's modulus, I is the second moment of the cross-sectional area, and e is the length of the link.

Similarly, the stiffness associated with shear deformation is given by

$$K_s = \frac{GA_w}{e} \tag{C5-40}$$

where G is the shear modulus and $A_w = t_w(d_b - 2t_f)$ is the area of the web. The ratio of bending to shear stiffness, $\beta = K_b/K_s$, characterizes the importance of shearing deformation to the stiffness. The stiffness of the link can be expressed in terms of β and the combined stiffness K given by

$$K = \frac{K_b K_s}{K_b + K_s} = \frac{K_b}{1 + \beta} \tag{C5-41}$$

The stiffness coefficients associated with unit rotation of one end, and unit translation of one end, of a link are given in Figure C5-19. It should be noted that for long beams, $\beta \to 0$ and the stiffness coefficients are the customary values used in ordinary structural analysis. When analyzing an EBF with a structural analysis program, the effects of shearing deformations must be accounted for by the program.

For a short link, energy associated with overloading is dissipated primarily through inelastic shearing of the link web. For a long link, the overload energy is dissipated primarily through plastic hinging at the ends of the link. The shear yielding energy dissipation mechanism is more efficient than the flexural plastic hinging mechanism.

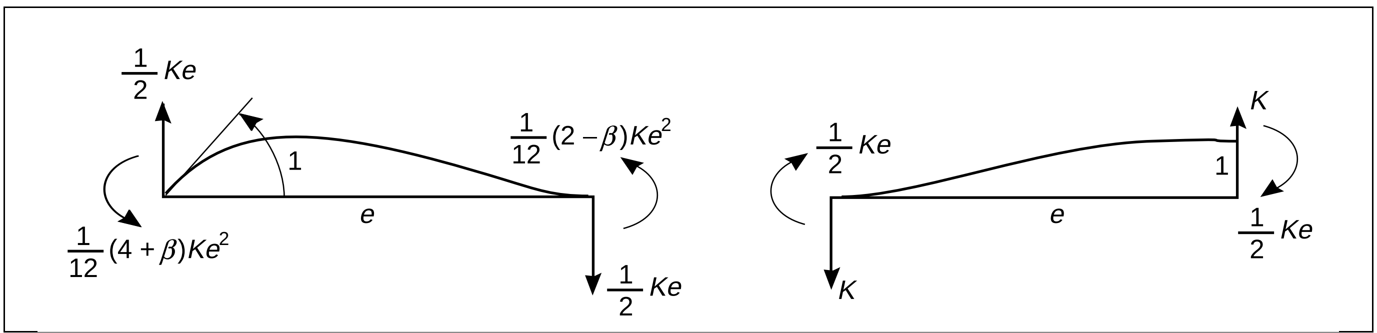


Figure C5-19 Stiffness Coefficients for a Link of Length e

The plastic capacity of a link is governed by shear-moment interaction. For design purposes, the shear-

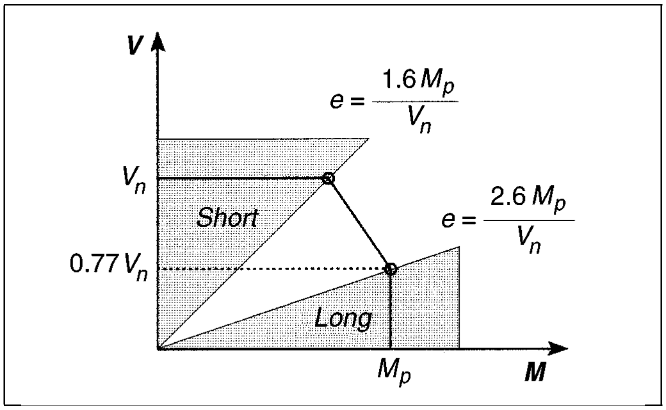


Figure C5-20 Shear-Moment Interaction

moment interaction diagram is idealized as shown in Figure C5-20. The nominal moment capacity of a beam is given by

$$M_p = F_y Z$$

where F_y is the uniaxial yield strength of the material and Z is the plastic section modulus. The nominal shear yield strength of a beam is given by

$$V_n = 0.6 F_y A_w$$

where $0.6F_y$ is the shear yield strength and $A_w = T_w = t_w(d_b - 2t_f)$ is the area of the web. These values provide the bounds on moment and shear that a link can sustain, as illustrated in the shear-moment interaction diagram of Figure C5-20. Moment M, shear V, and link length e are related through static equilibrium. The radial lines that emanate from the origin of the moment-shear interaction plot represent equilibrium lines for constant values of e.

The values $1.6M_p/V_n$ and $2.6M_p/V_n$ that define the bounds of short and long links in Figure C5-20 are based upon empirical observations. These different regions of link behavior are important to the following issues: (1) placement and detailing of web and flange stiffeners in the link region, (2) the strength of the link element, and (3) the ductility that the link element can supply. For short links, web buckling is the primary concern, while for long links local flange buckling is important. The requirements for placement and detailing of stiffeners can be found in Section 10.3 of AISC (1994a).

For a short link, the web yields while the flanges remain elastic. Therefore, the plastic capacity of a short link does not depend upon the moment carried by the link, and hence the shear capacity is $Q_{CE} = V_n$. A long link yields through the formation of a plastic hinge. The influence of the shear stresses on the yielding is so small that they do not affect the strength of the link. As the link yields, the forces tend to redistribute so that the full plastic moment develops on both ends of the link. Static equilibrium insists that $V = 2M_p/e$. Thus, the shear capacity can be equivalently expressed as $Q_{CE} = 2M_p/e$. The smallest link length that can be considered a long link is $e = 2.6M_p/V_n$. The shear capacity for a link of this length is therefore $Q_{CE} = 0.77V_n$. The capacity of a link of intermediate length is given by linear interpolation between the limiting values of short and long links; that is,

$$Q_{CE} = \left[1.37 - 0.23\frac{eV_{CE}}{M_{CE}}\right]V_{CE} \qquad \text{(C5-42)}$$

for $1.6 < EV_n/M_p < 2.6$.

The deformation of a link beam is characterized in terms of the angle between the axis of the link and the axis of the beam adjacent to the link, as shown in Figure C5-21. The link deformation angle at first yield can be computed as the shear force divided by the stiffness

$$\gamma_y = \frac{Q_{CE}}{Ke}$$

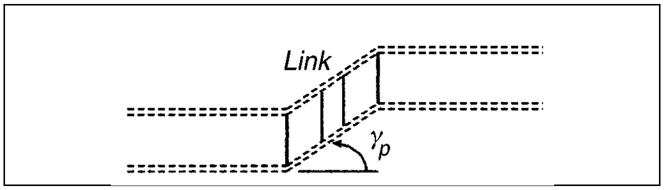


Figure C5-21 Link Rotation Angle

C5.5.3.3 Strength and Deformation Acceptance Criteria

The deformation capacity, γ_p, of a link beam depends upon the length of the link as well as the web and flange stiffening details. An idealization of link behavior is shown in Figure C5-22. The limit state for γ_p is web or flange buckling, as significant deterioration of link behavior begins after buckling. For adequately stiffened short links, the rotation capacity is approximately $\gamma_p = 0.12 \text{ rad}$.

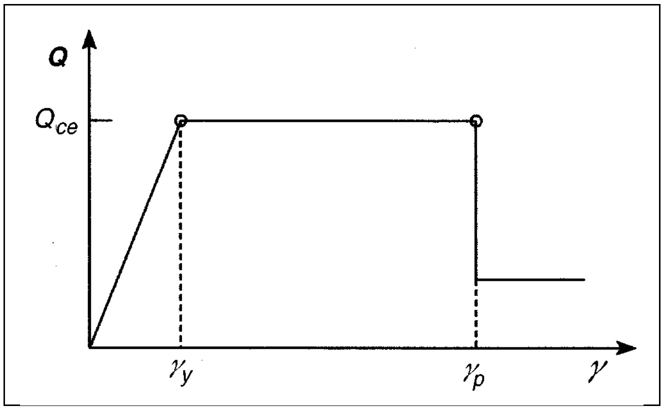


Figure C5-22 Deformation Capacity Definitions for a Link

Among reports giving experimental results are Ricles and Popov, 1987 and 1989; Hjelmstadt and Popov, 1983; Yang, 1982; Malley and Popov, 1983; Nishiyama et al., 1989; Whittaker et al., 1987 and 1989; Popov and Ricles, 1988; Foutch, 1989; and Foutch et al., 1987.

C5.5.3.4 Rehabilitation Measures for Eccentric Braced Frames

No commentary is provided for this section.

C5.6 Steel Plate Walls

No commentary is provided for this section.

C5.7 Steel Frames with Infills

The stiffness and resistance provided by concrete and/or masonry infills may be much larger than the stiffness of the steel frame acting alone with or without composite action. However, gaps or incomplete contact between the steel frame and the infill may negate some or all of this stiffness. These gaps may be between the wall and columns of the frame or between the wall and the top beam enclosing the frame. Different strength and stiffness conditions must be expected with different discontinuity types and locations. Therefore, the presence of any gaps or discontinuities between the infill walls and the frame must be determined and considered in the design and rehabilitation process. The resistance provided by infill walls may also be included if proper evaluation of the connection and interaction between the wall and the frame is made and if the strength, ductility, and properties of the wall are properly included.

Frames Attached to Masonry Walls. Attached walls are by definition somewhat separate from the steel frame. The stiffness and resistance provided by the walls may be large. However, the gaps or incomplete contact known to exist between the steel frame and the wall negate some or all of this strength and stiffness. As a result, the stiffness provided by attached masonry walls is excluded from the design and rehabilitation process unless integral action between the steel frame and the wall is verified. If complete or partial interaction between the wall and frame is verified, the stiffness is increased accordingly. The seismic performance of unconfined masonry walls is far inferior to that of confined masonry walls; therefore, the resistance of the attached wall can be used only if strong evidence as to its strength, ductility, and interaction with the steel frame is provided.

C5.8 Diaphragms

C5.8.1 Bare Metal Deck Diaphragms

C5.8.1.1 General

Diaphragms for bare steel decks are typically composed of corrugated sheet steel of 22 gage to 14 gage. The depths of corrugated sheet steel ribs vary from 1-1/2 to 3 inches in most cases, and attachment of the diaphragm to the steel frame occurs through puddle welds to the deck, typically at a spacing of one to two feet on center. This type of diaphragm is typically used only for roof construction. For large roof structures, supplementary diagonal bracing may be present for additional support.

The distribution of forces for existing diaphragms for bare steel decks is generally based on the flexible diaphragm assumption. Flexibility factors for various available types of diaphragms are available from manufacturers' catalogs. For systems where values are not available, it is best to interpolate with similar systems that do have values.

For bare metal decks, interaction between new and existing elements of the diaphragms (stiffness compatibility) must be considered as well as interaction with existing frames. Load transfer mechanisms between new and existing diaphragm elements and existing frames may need to be considered in flexibility of the diaphragm. (Analyses need to verify that diaphragm strength is not exceeded, so that elastic assumptions are still relatively valid.)

C5.8.1.2 Stiffness for Analysis

Inelastic properties of diaphragms are generally not included in inelastic seismic analyses. This is because diaphragm strength is generally quite high compared to demands, especially when concrete topping is present.

More flexible diaphragms, such as bare metal deck, 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 the weak link of the diaphragm is connector failure, then the element nonlinearity obviously cannot be incorporated into the model.

C5.8.1.3 Strength and Deformation Acceptance Criteria

Among the deficiencies most commonly found in bare metal deck diaphragms are:

- Inadequate connection between metal deck and chord or collector components
- Inadequate strength of chord or collector components
- Inadequate attachment of deck to supporting members
- Inadequate strength and/or stiffness of metal deck

C5.8.1.4 Rehabilitation Measures

Typical methods for correcting deficiencies in bare metal decks include:

- Adding shear connectors for chord or collector forces
- Strengthening existing chords or collectors by the addition of new steel plates to existing frame components
- Adding puddle welds or other shear connectors at panel perimeters
- Adding diagonal steel bracing to supplement diaphragm strength
- Replacing nonstructural fill with structural concrete
- Adding connections between deck and supporting members

New bare metal deck diaphragms should be designed and constructed in accordance with the recommendations of the Steel Deck Institute (SDI), given in the SDI Diaphragm Design Manual.

C5.8.2 Metal Deck Diaphragms with Structural Concrete Topping

C5.8.2.1 General

No commentary is provided for this section.

C5.8.2.2 Stiffness for Analysis

No commentary is provided for this section.

C5.8.2.3 Strength and Deformation Acceptance Criteria

Deficiencies that have been identified for metal deck diaphragms with structural concrete topping include:

- Inadequate connection between metal deck and chord or collector components (puddle welds and/or shear studs)
- Inadequate strength of chord or collector components
- Inadequate attachment of deck and concrete to supporting members
- Inadequate strength and/or stiffness of metal deck and composite concrete fill

C5.8.2.4 Rehabilitation Measures

Typical methods for correcting deficiencies include:

- Adding shear connectors for chord or collector forces

- Strengthening existing chords or collectors by the addition of new steel plates to existing frame components; also, attaching new plates directly to the slab with attachments such as embedded bolts, or epoxy
- Adding diagonal steel bracing to supplement diaphragm strength

New metal deck diaphragms with structural concrete topping should be designed and constructed in accordance with SDI recommendations or manufacturers' catalogs. Also, diaphragm shear capacity can be calculated considering the strength of concrete above the deck ribs in accordance with UBC or ICBO reports.

C5.8.3 Metal Deck Diaphragms with Nonstructural Concrete Topping

C5.8.3.1 General

No commentary is provided for this section.

C5.8.3.2 Stiffness for Analysis

No commentary is provided for this section.

C5.8.3.3 Strength and Deformation Acceptance Criteria

Deficiencies that have been identified for metal deck diaphragms with nonstructural concrete topping include

- Inadequate connection between metal deck and chord or collector components
- Inadequate strength of chord or collector components
- Inadequate attachment of deck to supporting members
- Inadequate strength and/or stiffness of metal deck and nonstructural concrete fill

C5.8.3.4 Rehabilitation Measures

Typical methods for correcting deficiencies in metal decks with nonstructural topping include

- Adding shear connectors for chord or collector forces
- Strengthening existing chords or collectors by the addition of new steel plates to existing frame elements, or attaching new plates directly to the slab with embedded bolts or epoxy
- Add puddle welds at panel perimeters of bare deck diaphragms
- Adding diagonal steel bracing to supplement diaphragm strength
- Replacing nonstructural fill with structural concrete

New metal deck diaphragms with structural concrete topping should be designed and constructed in accordance with SDI recommendations or manufacturers' catalogs. Also, diaphragm shear capacity can be calculated considering the strength of concrete above the deck ribs in accordance with UBC or ICBO reports.

C5.8.4 Horizontal Steel Bracing (Steel Truss Diaphragms)

C5.8.4.1 General

Horizontal steel trusses are generally used in combination with bare metal deck roofs or conditions where diaphragm stiffness is inadequate to transfer shear forces. It is more common for long spans or in situations with a longer overall width of diaphragm. Other examples are special roof structures of exposition halls, auditoriums, and others. The addition of horizontal steel trusses is one enhancement technique for weaker diaphragms.

The size and mechanical properties of the tension rods, compression struts, and connection detailing are all important to the yield capacity of the horizontal truss. Standard truss analysis techniques can be used to determine the yield capacity of the horizontal truss. Special attention is required at connections between different members of the horizontal truss. Connections that will develop the yield capacity of the truss members and reduce the potential for brittle failure are desired.

Stiffness can vary with different systems, but is most often fairly flexible with a fairly long period of vibration. Classical deflection analysis procedures can be used to determine the stiffness of the horizontal truss. Span-to-depth ratios of the truss system can have a significant effect on the stiffness of the horizontal truss. Lower span-to-depth ratios will result in increased stiffness of the horizontal truss. For equivalent lateral-force methods, factoring of the lateral force will be required to predict the actual deflection of the truss system.

More flexible, lower-strength horizontal truss systems may perform well for upgrades to the Life Safety Performance Level. Upgrades to the Damage Control Performance Range or the Immediate Occupancy Performance Level will require proportional increases in yield capacity and stiffness to control lateral displacements. Displacements must be compatible with the type of construction supported by the horizontal truss system.

Chord and collector elements for the above-listed diaphragms are generally considered to be composed of the steel frame elements attached to the diaphragm. For diaphragms with structural concrete, special slab reinforcement may be used in combination with the frame elements to make up the chords and/or collectors. The load transfer to the frame elements, which act as chords or collectors in modern frames, is generally through shear connectors. In older construction, the load transfer is made through bond when the frame is encased for fire protection.

C5.8.4.2 Stiffness for Analysis

Inelastic behavior may not be generally permitted in a steel truss diaphragm. Deformation limits to be established are to be more consistent with that of a diaphragm.

Classical truss analysis methods can be used to determine which members or connections of the existing horizontal truss require enhancement. Analysis of existing connections, and enhancement of connections with insufficient yield capacity, should be performed in a manner that will encourage yielding in the truss members rather than brittle failure in the truss connections.

C5.8.4.3 Strength and Deformation Acceptance Criteria

No commentary is provided for this section.

C5.8.4.4 Rehabilitation Measures

Deficiencies that may occur in existing horizontal steel bracing include the following:

- Various components of the bracing may not have strength to transfer all of the required forces.
- Various components of the bracing may not have sufficient ductility.
- Bracing connections may not be able to develop the strength of the members, or an expected maximum load.
- Bracing may not have sufficient stiffness to limit deformations below acceptable levels.

Typical methods for correcting deficiencies include the following:

- Diagonal components can be added to form a horizontal truss; this may be a method of strengthening a weak existing steel-framed floor diaphragm.
- Existing chord components may be strengthened by the addition of shear connectors to enhance composite action.
- Existing steel truss components may be strengthened by methods similar to those noted for braced steel frame members.
- Truss connections may be strengthened by the addition of welds, new or enhanced plates, and bolts.
- Where possible, structural concrete fill may be added to act in combination with steel truss diaphragms. Gravity load effects of the added weight of the concrete fill must be considered in such a solution.

Design of completely new horizontal steel bracing elements should generally follow the procedures required for new braced frame elements.

C5.8.5 Archaic Diaphragms

C5.8.5.1 General

No commentary is provided for this section.

C5.8.5.2 Stiffness for Analysis

No commentary is provided for this section.

C5.8.5.3 Strength and Deformation Acceptance Criteria

No commentary is provided for this section.

C5.8.5.4 Rehabilitation Measures

Deficiencies that may occur in existing archaic diaphragms include the following:

- The lack of steel reinforcing severely limits the ability of the element to resist diagonal tension forces without significant cracking.
- Diagonal tension could jeopardize the compression forces in the brick arches, creating a situation that could lead to loss of support.
- Connections between the brick work and steel may not be able to transfer the required diaphragm forces.
- The diaphragm may not have sufficient stiffness to limit deformations below acceptable levels.

Typical methods for correcting deficiencies include the following.

- Diagonal elements can be added to form a horizontal truss.
- Existing steel members may be strengthened by the addition of shear connectors to enhance composite action.
- Weak concrete fill may be removed and replaced by a structural reinforced concrete topping slab. Gravity load effects of the added weight of the concrete fill must be considered in such a solution.

C5.8.6 Chord and Collector Elements

C5.8.6.1 General

No commentary is provided for this section.

C5.8.6.2 Stiffness for Analysis

No commentary is provided for this section.

C5.8.6.3 Strength and Deformation Acceptance Criteria

No commentary is provided for this section.

C5.8.6.4 Rehabilitation Measures

Deficiencies that have been identified for chords and collectors include:

- Inadequate connection between diaphragm and chords or collectors
- Inadequate strength of chord or collector
- Inadequate detailing for strength at openings or re-entrant corners

Typical methods for correcting deficiencies include the following:

- The connection between diaphragms and chords and collectors can be improved.
- Chords or collectors can be strengthened with steel plates. New plates can be attached directly to the slab with embedded bolts or epoxy. Also, reinforcing bars can be added to the slab.
- A structural slab can be added to improve compressive capacity of existing chords and collectors.
- Chord members can be added.

New chord and collector components should be designed in accordance with the requirements of the AISC Manual or ACI Building Code.

C5.9 Steel Pile Foundations

C5.9.1 General

No commentary is provided for this section.

C5.9.2 Stiffness for Analysis

Two analytical models are commonly used to analyze pile foundations: the equivalent soil spring model and the equivalent cantilever model. These are shown schematically in Figure C5-23.

The equivalent soil spring model is often used for the design of pile foundations for bridges. The properties of the soil spring are dependent on the soil properties at the site. Both linear and nonlinear models are available. A complete description of the model and a computer program for its implementation are given in FHWA (1987).

Before the development of the equivalent soil spring model, the primary model used to obtain the stiffness and maximum moments for piles was the equivalent cantilever method, represented in Figure C5-24. The pile is considered to be a cantilever column. The stiffness of the pile is assumed to be the same as for a free-standing cantilever column with a length of L_s. The maximum moment in the pile is assumed to be the same as for a free-standing cantilever column with a length of L_M. The lengths L_S and L_M depend on EI of the pile and a soil constant as given in Figure C5-24. Additional information on pile capacity may be found in Davisson (1970) and in most foundation engineering textbooks.

C5.9.3 Strength and Deformation Acceptance Criteria

In most situations the calculation of the pile strength is straightforward, since buckling is not a consideration unless the pile extends above the ground surface or through a liquefiable soil. A pile that extends above the ground surface may be analyzed as a free-standing column with length $L_C = (L_F + L_S)$ and $K = 1.0$ where L_C is the equivalent column length, L_F is the length above ground, and L_S is as given in Figure C5-24. For piles that pass through a liquefiable soil, guidance should be sought from a geotechnical engineer.

C5.9.4 Rehabilitation Measures for Steel Pile Foundations

No commentary is provided for this section.

C5.10 Definitions

No commentary is provided for this section.

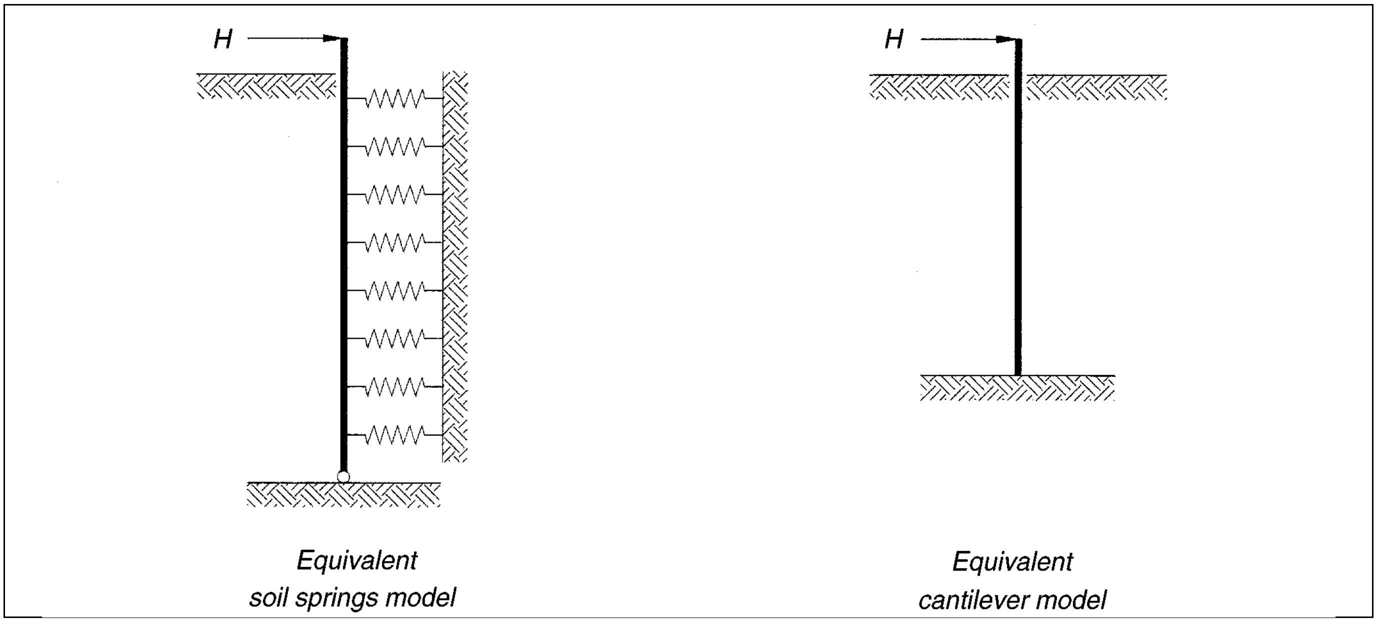
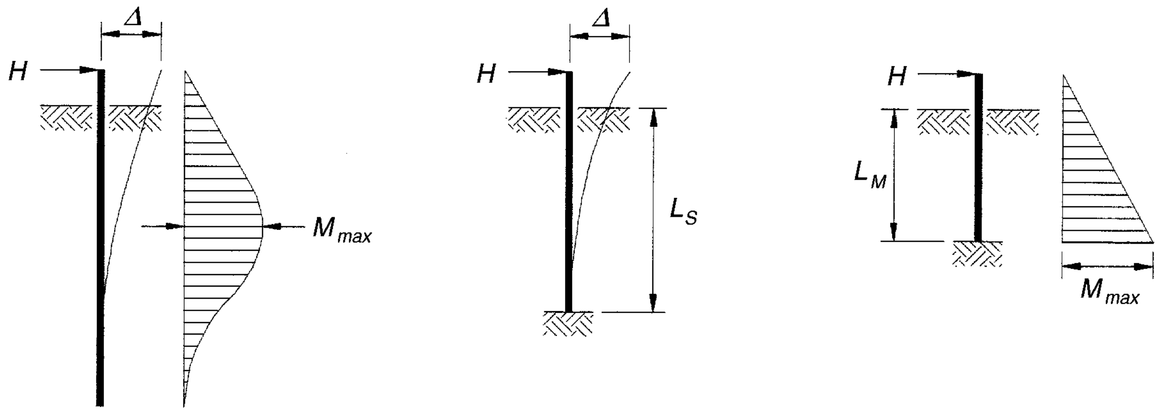


Figure C5-23 Models for Pile Analysis

C5.11 Symbols



	L_S	L_M
Cohesive soil constant K_h	$1.4\sqrt[4]{\frac{EI}{K_h}}$	$0.44\sqrt[4]{\frac{EI}{K_h}}$
Cohesionless soil constant n_h	$1.8\sqrt[5]{\frac{EI}{n_h}}$	$0.78\sqrt[5]{\frac{EI}{n_h}}$

Figure C5-24 Equivalent Cantilever Model for Piles

This list may not contain symbols defined at their first use if not used thereafter.

A_c Gross cross-sectional area of connector, in.^{2}

A_e Net effective area of stem, in.^{2}

A_g Gross area of T-stub stem, in.^{2}

A_w Area of web of link beam, in.^{2}

E Modulus of elasticity, 29,000 ksi

F_{ve} Expected shear strength of connector, ksi

F_y Yield strength, ksi

F_{ye} Expected yield strength, ksi

G Shear modulus, ksi

I_b Moment of inertia of beam, in.^{4}

$I_b adj$ Adjusted moment of inertia of beam, in.^{4}

I_c Moment of inertia of column, in.^{4}

K Stiffness of a link beam, kip/in.

K Coefficient for Equations C5-9, C5-25, and C5-38

K_b Flexural stiffness of link beam, kip-in./rad

K_θ Rotational stiffness of a partially-restrained connection, kip-in./rad

M_{CE} Expected flexural strength of a member or joint, kip-in.

M_{CE} Expected flexural strength, kip-in.

N_{OSL} Number of connectors in outstanding leg of clip angle, dimensionless

N_{stem} Number of connectors in stem of T-stub connection, dimensionless

N_{VL} Number of tensile connectors in T-stub connection, dimensionless

P Force, kips

P_{CE} Expected strength, kips

Q_{CE} Effective expected shear strength of link beam, kips

Z Plastic section modulus, in.^{3}

d Dimension of end plate connection, in.

d_b Beam depth, in.

f Bolt diameter, in.

h Story height, in.

k_s Rotational stiffness of connection, kip-in./rad

k_s Shear stiffness of link beam, kip/in.

l_b Length of beam, in.

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

t Plate thickness, in.

t_f Flange thickness, in.

t_s Stem thickness of T-stub, in.

t_w Thickness of web of link beam, in.

u Deflection, in.

w Width of T-stub, in.

Δ Generalized deformation, dimensionless

γ_p Deformation capacity of link beam, radians

γ_y Yield deformation of link beam, radians

θ Rotation, radians

C5.12 References

AISC, 1994a, *Manual of Steel Construction, Load and Resistance Factor Design Specification for Structural Steel Buildings (LRFD), Volume I, Structural Members, Specifications and Codes*, American Institute of Steel Construction, Chicago, Illinois.

AISC, 1994b, *Manual of Steel Construction, Load and Resistance Factor Design, Volume II, Connections*, American Institute of Steel Construction, Chicago, Illinois.

Anderson, R. C., 1983, *Inspection of Metals, Volume I: Visual Inspection*, American Society for Metals International, Metals Park, Ohio.

Anderson, R. C., 1988, *Inspection of Metals, Volume 2: Destructive Testing*, American Society for Metals, International, Metals Park, Ohio.

Andreaus, U., and Gaudenzi, P., 1989, "Modeling of Cyclic Behavior of Steel Braces, Res Mechanica," *International Journal of Structural Mechanics and Materials Science*, Vol. 26, No. 3, pp. 267–288.

Aslani, F., and Goel, S. C., 1991, "Stitch Spacing and Local Buckling in Seismic-Resistant Double-Angle Braces," *Journal of the Structural Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 117, No. 8, pp. 2442–2463.

Aslani, F., and Goel, S. C., 1989, *Experimental and Analytical Study of the Inelastic Behavior of Double Angle Bracing Members Under Severe Cyclic Loading*, Report No. UMCE 89-5, Department of Civil Engineering, University of Michigan, Ann Arbor, Michigan.

Astaneh-Asl, A., and Goel, S. C., 1984, "Cyclic In-Plane Buckling of Double Angle Bracing," *Journal of the Structural Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 110, No. 9, pp. 2036–2055.

Astaneh-Asl, A., Goel, S. C., and Hanson, R. D., 1986, "Earthquake-Resistant Design of Double-Angle Bracings," *Engineering Journal of the American Institute of Steel Construction*, Chicago, Illinois, Vol. 23, No. 4, pp. 133–147.

Azizinamini, A., and Readziminski, J. B., 1989, "Static and Cyclic Performance of Semirigid Steel Beam-to-Column Connections," *Journal of the Structural Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 115, No. 12.

Bartlett, F. M., and Sexsmith, R. G., 1991, "Bayesian Technique for Evaluation of Material Strengths in Existing Bridges," *CI Materials Journal*, American Concrete Institute, Detroit, Michigan, Vol. 88, No. 2.

Batho, C., 1938, "The Effect of Concrete Encasement on the Behavior of Beam and Stanchion Connections," *The Structural Engineer*, Institute of Structural Engineers, London, England, Vol. 16, No. 12.

Batho, C., and Lash, S. D., 1936, *Further Investigations on Beam and Stanchion Connections, Including Connections Encased in Concrete: Together with Laboratory Investigations on a Full-Scale Frame*, Continuation of the Second Report of the Steel Structures Research Committee, Her Majesty's Stationery Office, London, England, pp. 276–363.

Bertero, V. V., Uang, Chia-M., Llopiz, C. R., and Igarashi, K., 1989, “Earthquake Simulator Testing of Concentric Braced Dual System,” *Journal of the Structural Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 115, No. 8, pp. 1877–1894.

Bertero, V. V., Uang, Chia-M., and Whittaker, A. S., 1989, “Earthquake Simulator Testing of a Concentrically Braced Steel Structure,” *Proceedings, Ninth World Conference on Earthquake Engineering*, 9WCEE Organizing Committee, Japan Association for Earthquake Disaster Prevention, Tokyo, Japan, Vol. VIII, Paper SI-8, pp. 883–888.

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 (Reports No. FEMA 222A and 223A), Washington, D.C.

Campbell, H. H., 1895, “Specifications for Structural Steel,” *Transactions*, American Society of Civil Engineers, New York, New York, Vol. XXXIII, p. 297.

Davisson, M. T., 1970, *Lateral Load Capacity of Piles*, Highway Research Record No. 323 - Pile Foundations, Highway Research Board.

El-Tayem, A., and Goel, S. C., 1986, “Effective Length Factor for the Design of X-Bracing Systems,” *AISC Engineering Journal*, American Institute of Steel Construction, Chicago, Illinois, Vol. 23, No. 1, pp. 41–45.

FHWA, 1987, *COM624P Program Users Manual Version 1.0*, Report No. FHWA-SA-91-502, Federal Highway Administration, Washington, D.C.

FitzGerald, T. F., Anagnos, T., Goodson, M., and Zsutty, T., 1989, “Slotted Bolted Connections in Aseismic Design for Concentrically Braced Connections,” *Earthquake Spectra*, Earthquake Engineering Research Institute, Oakland, California, Vol. 5, No. 2, pp. 383–391.

Foutch, D. A., 1989, “Seismic Behavior of Eccentrically Braced Steel Building,” *Journal of the Structural Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 115, No. 8, pp. 1857–1876.

Foutch, D. A., Goel, S. C., and Roeder, C. W., 1987, “Seismic Testing of Full-Scale Building, Part I and Part II,” *Journal of the Structural Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 112, No. 11, pp. 2111–2129.

Freitag, J. K., 1906, *Architectural Engineering*, Second Edition, John Wiley & Sons, New York, New York.

Frye, J. M., and Morris, G. A., 1975, “Analysis of Flexibly Connected Steel Frames,” *Canadian Journal of Civil Engineering*, National Research Council of Canada, Ottawa, Canada, Vol. 2, pp. 280–291.

Fukuta, T., Yamanouchi, H., Nishiyama, I., Endott, N., and Watanabe, T., 1984, “Hysteresis Behavior of Three Story Concentric K-Braced Frames,” *Proceedings of the Eighth World Conference on Earthquake Engineering*, San Francisco, California, Earthquake Engineering Research Institute, Oakland, California, pp. 233–240.

Fukuta, T., et al., 1989, “Seismic Performance of Steel Frames with Inverted V Braces,” *Journal of the Structural Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 115, No. 8, Special Issue: U.S./Japan Studies of Steel Structures, pp. 2016–2028.

Goel, S. C., 1986, “Seismic Stability of Braced Steel Structures,” *Proceedings—Annual Technical Session: Stability of Plate Structures*, Washington, D.C., Structural Stability Research Council, Bethlehem, Pennsylvania, pp. 189–200.

Goel, S. C., and Astaneh-Asl, A., 1984, “Cyclic Behavior of Double Angle Bracing Members with Welded Connections,” *Proceedings of Eighth World Conference on Earthquake Engineering*, San Francisco, California, Earthquake Engineering Research Institute, Oakland, California, pp. 241–248.

Goel, S. C., and El-Tayem, A. A., 1986, “Cyclic Load Behavior of Angle X-Bracing,” *Journal of the Structural Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 112, No. 11,
pp. 2528–2539.

Gugerli, H., and Goel, S. C., 1982, *Inelastic Cyclic Behavior of Steel Bracing Members*, Report No. UMEE

82R1, Department of Civil Engineering, University of Michigan, Ann Arbor, Michigan.

Harriott, J. D., and Astaneh-Asl, A., 1990, *Cyclic Behavior of Steel Top-and-Bottom Plate Moment Connections*, Report No. UCB/EERC-90-19, Earthquake Engineering Research Center, University of California, Berkeley, California.

Hassan, O., and Goel, S. C., 1991, *Modeling of Bracing Members and Seismic Behavior of Concentrically Braced Steel Structures*, Report No. UMCE 91-1, Department of Civil Engineering, University of Michigan, Ann Arbor, Michigan.

Hechtman, R. A., and Johnston, B. G., 1947, *Riveted Semi-Rigid Beam-to-Column Building Connections*, Progress Report No. 1, Committee of Steel Structures Research, American Institute of Steel Construction, Chicago, Illinois.

Hertzverg, R. W., 1985, *Deformation and Fracture Mechanics in Engineered Materials*, John Wiley & Sons, New York, New York.

Hjelmstadt, K. D., and Popov, E. P., 1983, S*eismic Behavior of Active Beam Links in Eccentrically Braced Frames*, Report No. UCB/EERC–83/15, Earthquake Engineering Research Center, University of California, Berkeley, California.

Ikeda, K., and Mahin, S. A., 1986, "Cyclic Response of Steel Braces," *Journal of the Structural Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 112, No. 2, pp. 342–361.

Ikeda, K., and Mahin, S. A., 1984, *Refined Physical Theory Model for Predicting the Seismic Behavior of Braced Steel Frames*, Report No. UCB/EERC-84/12, Earthquake Engineering Research Center, University of California, Berkeley, California.

Jain, A. K., and Goel, S. C., 1978, *Inelastic Cyclic Behavior of Bracing Members and Seismic Response of Braced Frames of Different Proportions*, Report No. UMEE78R3, Department of Civil Engineering, University of Michigan, Ann Arbor, Michigan.

Jefferson, T. B., 1990, *Metals and How to Weld Them*, Second Edition, The James F. Lincoln Arc Welding Foundation, Cleveland, Ohio.

Johnstone, N. D., and Walpole, W. R., 1981, *Bolted End-Plate Beam-to-Column Connections Under Earthquake Type Loading*, Research Report 81-7, Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand.

Kariotis, J., 1991, "Testing of Archaic Materials for Determination of Seismic Resistance Capacities," *The Seismic Retrofit of Historic Buildings Conference Workbook*, Western Chapter of the Association for Preservation Technology, San Francisco, California.

Ketchum, M. S., 1918, *Structural Engineer's Handbook*, McGraw-Hill Book Company, New York, New York.

Khatib, I. F., Austin, M. A., and Mahin, S. A., 1987, "Dynamic Inelastic Behavior of Chevron Braced Steel Frames," *Earthquake Engineering: Fifth Canadian Conference*, Proceedings: Ottawa, A.A. Balkema, Rotterdam, The Netherlands, pp. 211–220.

Khatib, I. F., Mahin, S. A., and Pister, K. S., 1988, *Seismic Behavior of Concentrically Braced Steel Frames*, Report No. UCB/EERC-88/01, Earthquake Engineering Research Center, University of California, Berkeley, California.

Kulak, G. L., Fisher, J. W., and Struik, J.H.A., 1987, *Guide to Design Criteria for Bolted and Riveted Joints*, Second Edition, John Wiley & Sons, New York, New York.

Lee, S., and Goel, S. C., 1987, *Seismic Behavior of Hollow and Concrete-Filled Square Tubular Bracing Members*, Report No. UMCE 87-11, Department of Civil Engineering, University of Michigan, Ann Arbor, Michigan.

Liu, Z., and Goel, S. C., 1988, "Cyclic Load Behavior of Concrete-Filled Tubular Braces," *Journal of the Structural Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 114, No. 7, pp. 1488–1506.

Lorenz, P. G., 1990, *The Science of Remote Visual Inspection (RVI): Technology, Applications, Equipment*, American Society for Nondestructive Testing, Columbus, Ohio.

Malley, J. O., and Popov, E. P., 1983, *Design Considerations for Shear Links in Eccentrically Braced*

Frames, Earthquake Engineering Research Center, College of Engineering, University of California, Berkeley, California.

Mathey, R. G., and Clifton, J. R., 1988, *Review of Nondestructive Evaluation Methods Applicable to Construction Materials and Structures*, NBS Technical Note 1247, U.S. Department of Commerce, National Bureau of Standards, Washington, D.C.

Midorikawa, M., Nishiyama, I., and Yamanouchi, H., 1989, "Static and Dynamic Analysis of Full-Scale Concentrically K-Braced Steel Building," *Proceedings, Ninth World Conference on Earthquake Engineering*, 9WCEE Organizing Committee, Japan Association for Earthquake Disaster Prevention, Tokyo, Japan, Vol. IV, Paper 6-3-9, pp. 255–260.

Murray, T. M., and Kukreti, A. R., 1988, "Design of 8-Bolt Stiffened Moment End Plate," *Engineering Journal*, American Institute of Steel Construction, Vol. 25, No. 2, Chicago, Illinois.

Nishiyama, I., Midorikawa, M., and Yamanouchi, H., 1989, "Inelastic Behavior of Full-Scale Eccentrically K-Braced Steel Building," *Proceedings, Ninth World Conference on Earthquake Engineering*, 9WCEE Organizing Committee, Japan Association for Earthquake Disaster Prevention, Tokyo–Kyoto, Japan, Vol. IV, pp. 261–266.

Paulson, C., Tide, R.H.R., and Meinheit, D. F., 1994, *Modern Techniques for Determining the Capacity of Cast Iron Columns*, ASTM Special Technical Publication 1258: Standards for Preservation and Rehabilitation, edited by S. J. Kelly, American Society for Testing and Materials, Philadelphia, Pennsylvania.

Popov, E. P., and Pinkney, R. B., 1969, "Cyclic Yield Reversal in Steel Building Connections," *Journal of the Structural Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 95, No. ST3.

Popov, E., Ricles, P., and James, M., 1988, *Experimental Study of Seismically Resistant Eccentrically Braced Frames with Composite Floors*, Report No. UCB-EERC-88-17, Earthquake Engineering Research Center, University of California, Berkeley, California, pp. 149–154.

Rai, D., Goel, S. C., and Firmansjah, J., 1995, *SNAP-2DX—A General Purpose Computer Program for Static and Dynamic Nonlinear Analysis of 2-D Structures: User's Guide*, Department of Civil & Environmental Engineering, University of Michigan, Ann Arbor, Michigan.

Rathbun, J. C., 1936, *Elastic Properties of Riveted Connection*s, Transactions of American Society of Civil Engineers, New York, New York, Vol. 101, pp. 524–563.

Redwood, R. G., Lu, F., Bouchard, G., and Paultre, P., 1991, "Seismic Response of Concentrically Braced Steel Frames," *Canadian Journal of Civil Engineering*, National Research Council of Canada, Ottawa, Canada, Vol. 18, No. 6, pp. 1062–1077.

Ricles, J. M., and Popov, E. P., 1987, *Experiments on Eccentrically Braced Frames with Composite Floors*, Report No. UCB/EERC–87/06, Earthquake Engineering Research Center, University of California, Berkeley, California.

Ricles, J. M., and Popov, E. P., 1989, "Composite Action in Eccentrically Braced Frames," *Journal of the Structural Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 115, No. 8, pp. 2046–2066.

Roeder, C. W., 1989, "Seismic Behavior of Concentrically Braced Frames," *Journal of the Structural Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 115, No. 8, pp. 1837–1856.

Roeder, C. W., Leon, R. T., and Preece, F. R., 1994, *Strength, Stiffness and Ductility of Older Steel Structures Under Seismic Loading*, SGEM Report 94-4, Department of Civil Engineering, University of Washington, Seattle, Washington.

Roeder, C. W., and Foutch, D. A., 1996, "Experimental Results for Seismic Resistant Steel Moment Frame Connections," *Journal of the Structural Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 122, No. 6, pp. 581–588.

SAC, 1995, *Interim Guidelines: Evaluation, Repair, Modification and Design of Welded Steel Moment Frame Structures*, developed by the SEAOC, ATC, and CUREE Joint Venture (Report No. SAC–95–02) for the

Federal Emergency Management Agency (Report No. FEMA 267), Washington, D.C.

Schneider, Charles C., 1905, “The Structural Design of Buildings,” *Transactions*, American Society of Civil Engineers, New York, New York, Vol. LIV, P. 371, paper 997.

Sherbourne, A. N., June 1961, “Bolted Beam-to-Column Connections,” *The Structural Engineer*, Institute of Structural Engineers, London, England, Vol. 39, pp. 203–210.

Takanashi, K., and Ohi, K., 1984, “Shaking Table Tests on 3-Story Braced and Unbraced Steel Frames,” *Proceedings of the Eighth World Conference on Earthquake Engineering*, Earthquake Engineering Research Institute, Oakland, California, Vol. VI, pp. 193–200.

Tang, X., and Goel, S. C., 1987, *Seismic Analysis and Design Considerations of Concentrically Braced Steel Structures*, Report No. UMCE 87-4, Department of Civil Engineering, University of Michigan, Ann Arbor, Michigan.

Tsai, K. D., and Popov, E. P., 1990, “Cyclic Behavior of End-Plate Moment Connections,” *Journal of the Structural Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 116, No. 11.

Uang, Chia-M., and Bertero, V. V., 1986, *Earthquake Simulation Tests and Associated Studies of a 0.3-Scale Model of a Six-Story Concentrically Braced Steel Structure*, Report No. UCB/EERC-86/10, University of California, Berkeley, California.

Whittaker, A. S., Uang, Chia-M., and Bertero, V. V., 1987, *Earthquake Simulation Tests and Associated Studies of a 0.3-Scale Model of a Six-Story Eccentrically Braced Steel Structure*, Report No. UCB/EERC-87/02, University of California, Berkeley, California.

Whittaker, A. S., Uang, Chia-M., and Bertero, V. V., 1989, “Seismic Testing of Eccentrically Braced Dual Steel Systems,” *Earthquake Spectra*, Earthquake Engineering Research Institute, Oakland, California, Vol. 5, No. 2, pp. 429–449.

Whittaker, D., and Walpole, W. R., 1982, *Bolted End-Plate Connections for Seismically Designed Steel Frames*, Research Report 82-11, Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand.

Wijanto, L. S., Moss, P. J., and Carr, A. J., 1992, “Seismic Behavior of Cross-Braced Steel Frames,” *Earthquake Engineering & Structural Dynamics*, Elsevier Science, New York, New York, Vol. 21, No. 4, pp. 319–340.

Wulpi, D. J., 1985, *Understanding How Components Fail*, American Society for Metals, International Metals Park, Ohio.

Xu, P., and Goel, S. C., 1990, *Behavior of Double Channel Bracing Members Under Large Cyclic Deformations*, Report No. UMCE 90-1, Department of Civil Engineering, University of Michigan, Ann Arbor, Michigan.

Yang, M. S., 1982, *Seismic Behavior of an Eccentrically X-Braced Steel Structure*, Report No. UCB/EERC-82/14, Earthquake Engineering Research Center, University of California, Berkeley, California.

Yang, M. S., 1984, “Shaking Table Studies of Eccentrically X-Braced Steel Structure,” *Proceedings of the Eighth World Conference on Earthquake Engineering*, San Francisco, California, Earthquake Engineering Research Institute, Oakland, California, pp. 257–264.

C6. Concrete (Systematic Rehabilitation)

C6.1 Scope

The scope of Chapter 6 is broad, in that it is intended to include all concrete structural systems and embedded connection components. Concrete masonry systems are covered in Chapter 7. Exterior concrete cladding is covered in Chapter 11.

Material presented in Chapter 6 is intended to be used directly with the Analysis Procedures presented in Chapter 3.

C6.2 Historical Perspective

This section covers a broad range of older existing reinforced concrete construction. A historical background is provided in the following paragraphs to aid in defining the scope, as well as to provide guidance on likely characteristics of existing construction. Tables 6-1 through 6-3 of the *Guidelines* also contain historical material properties, as illustrated in the following text.

History of Reinforced Concrete Materials. Concrete as material has engineering properties that are highly complex. Despite the complex nature of the material, the characteristics of concrete are usually summarized in terms of the compressive strength. It is assumed that other properties—such as the concrete contribution to shear strength, the elastic modulus, the shear modulus, and the tensile strength—are related to the compressive strength by standard relationships that are expressed in the provisions for design of new buildings. It has been found that this approach is suitable both for design of new buildings and for evaluation of existing buildings. No change in this approach is suggested.

Concrete compressive strengths have increased steadily over the years. Results of tests of cores from early buildings may be found to be highly variable, but typical maxima strengths are in the range of 2500–3000 psi. These values are consistent with those found in building codes of the time of construction, and in textbooks of the same era. Currently, these same values are the minimum that will be found in practice, and concrete strengths for routine cast-in-place construction generally are in the range of 4000–5000 psi, with considerable variation in different areas of the United States. Strengths of concrete in prestressed construction are generally specified in the range of 6,000–10,000 psi. Some specialized concretes, such as for columns in tall buildings, may be found with compressive strengths as high as 18,000 psi.

To the greatest extent possible, concrete structures should be inspected throughout for evidence of concrete that has properties different from the average or from test results that may have been obtained. This is particularly important for very early structures, or structures for which the test results have been very erratic. Visual evidence may include changes in color or consistency of the concrete, poor compaction, distress, or obvious deterioration.

Reinforcing bars also have shown a consistent increase in strength over the years. Early bars may be structural grade with a yield strength of 33,000 psi, while 60,000 psi yield is the current design standard. However, high-strength bars have been available for many years, from early hard grade bars with 50,000 psi yield to the current 75,000 psi yield.

Proprietary bar shapes used in early construction can be expected to have strengths similar to those of standard bars. These include shapes such as square bars, twisted bars, and plain round bars. Plain bars, without deformations, will often be found in early structures. Bond capacity values should be reduced accordingly (see Section C6.3).

Chronology of the Use of Reinforced Concrete in Buildings. The date of construction correlates with the architectural treatment, type of construction, construction methods, materials, and building codes. These factors in turn influence seismic performance, and must be considered in evaluation and design of retrofit measures. Types of construction and, to a certain extent, construction methods, are discussed in the following sections.

1900–1910. Construction of buildings using reinforced concrete began at about the start of the 20th century, as portland cement became commercially available and more individuals became familiar with its characteristics. As would be expected, the first buildings mimicked the structural systems common with other materials, so we find frame buildings with concrete columns, girders, beams, and slabs. Concrete bearing wall buildings are found as well, but these seem to be less common in early construction than the frame configuration.

Concrete in some early buildings may have been mixed by hand, batch by batch, in wheelbarrows immediately adjacent to where it would be placed in the structure. The resulting concrete would be highly variable in quality within very short distances in a structure—a possibility to be kept in mind in analyzing the strength of very early structures.

Exterior walls in frame buildings of this era commonly were either masonry infills in the plane of the frame, or curtain walls partially within the frame and partially outside it. Infill materials might be brick or concrete masonry, which are relatively strong but brittle, or clay tile stucco or terra cotta, which are weak and brittle. Exterior facing materials commonly were brick or stone masonry.

Most frame buildings constructed in this period had multiple interior partitions, which contributed to stiffness, strength (to a certain degree), and internal damping. Original construction materials included clay tile, lath (wood or metal) and plaster, or masonry. In the intervening years, these partitions may have been moved repeatedly, or removed without replacement. The replacements in recent years are likely to be gypsum board on wood or metal studs—a weaker, more flexible system, but much lighter. In many cases, the original partitions may not have been replaced at all, leaving an open floor plan. The resulting current configuration in many of these older buildings may be mixture of interior partitions of many types, with the accompanying variations in weight, stiffness, and strength, and with some partitions missing entirely. These variations may be within a floor, and between floors. The resulting eccentricities in mass and stiffness, and vertical variations, should be taken into account in the analysis process.

1910–1920. Dates for introduction of specific structural systems are always approximate, but it is fair to say that the development of specialized systems in cast-in-place concrete began about this time. A notable example is the flat slab floor system, which utilizes the heterogeneous nature of concrete to create a floor system more free of directional characteristics. The flat slab floor system consists of an array of columns, not necessarily on a rectangular grid, supporting a constant thickness floor that does not have beams. Most early examples were designed for heavy loads, so that it was necessary to thicken the floor in the vicinity of the columns. These thickened portions, called drop panels, provided increased moment and shear capacity. In many cases, enlargements of the tops of the columns, called capitals, were also provided.

These early flat slabs often were reinforced with proprietary systems using reinforcement arrangements that seem very strange when compared with current practice. Elaborate combinations of multiple directions of bars, interlocking circles, and other complex forms are found. The possibility of the presence of one of these systems should be considered if location of bars by electromagnetic means is being attempted in one of these early buildings. Similarly, reinforcing steel optimization became more attractive; continuity of bars at member connections must be carefully considered.

About this same time period, techniques for reduction of structural weight became of interest, particularly for buildings with lighter live loads. Concrete joist construction was developed, where in one direction the beam and slab construction became a constant depth arrangement of narrow, closely spaced (about 30 inches, typically) beams called joists, with very thin concrete slabs between them to complete the floor surface. The construction of the floor system is started by building a form work platform on which void formers are placed in the desired pattern. Reinforcement for the joists and slab are placed. Concrete is then cast around and above the void formers to create a ribbed slab with a smooth upper surface.

The void formers may be steel pans open on the bottom, or they may be hollow clay tiles, which would result in a smooth ceiling line. The smooth appearance may have been enhanced by a coat of hard plaster. As far as evaluation is concerned, the significance is that what appears to be solid concrete—and may sound like solid concrete when tapped lightly with a hammer—may actually be weak, brittle clay tile in some locations. Care should be taken to ensure that a proposed retrofit element bears on concrete, not on an area of concealed voids such as may be represented by the clay tile. Also, the additional weight of the masonry forming materials must be accounted for.

A variation on the concrete joist system is the waffle slab system. As the name implies, the joists run in perpendicular directions so that the crossing patterns leave square voids that appear on the underside not unlike a waffle pattern. Some early versions used clay tile left in place and plastered over on the bottom, so the above cautions about the same construction in concrete joists also apply for such waffle slabs. More recent examples—using metal pans or cardboard forms—leave the system exposed for architectural effect, which makes identification very easy.

All these structural systems are still in use for new construction, although clay tile void formers are no longer in use in the United States. It should be noted that the heavy, and relatively deep, floor systems are likely to create a strong beam-weak column situation that will be discussed further in conjunction with concrete construction.

About this same time period, use of concrete bearing walls became more common, particularly for industrial structures and for commercial structures built against lot lines. For the most part these would be low-rise structures. The walls may have very little reinforcement, and may not be adequately connected to the floors and roof diaphragm.

1920–1930. This period represented an era of improvement more than one of innovation. Construction became more mechanized, so the likelihood of encountering localized variations in concrete quality was reduced, although voids due to poor consolidation are a possibility.

By this period, sufficient time had elapsed since concrete construction had become common that weak points in performance could be identified and corrected, at least for response to gravity loads. Seismic design was in its infancy, so it is likely that any intentional lateral-force-resisting systems found during evaluations will be proportioned for wind forces only.

1930–1950. This period was dominated by external events, namely the Depression and World War II, so progress in concrete construction was slight. Research went on, to a degree, and some refinement continued in design and construction, but for the most part building types and construction methods changed little in this period. The level of construction, particularly in the Depression, was only a fraction of what it had been earlier. Construction activity increased during and after the war, but most research efforts and refinements in materials and construction techniques were directed elsewhere.

1950–1960. This period saw a very rapid change in building systems, design methods, and construction practice. As a result of problems associated with the increased rate of change, buildings built in this period may well require closer scrutiny than their counterparts built earlier. The use of deformed reinforcing steel became prominent during this period, displacing smooth and proprietary systems.

More open interiors, and the use of lightweight metal or glass curtain wall exterior cladding, meant that frame buildings had less stiffness, and possibly less initial strength as well. Coupled with the fact that design for lateral loads in general, and seismic loads in particular, had still not reached relative maturity, these buildings may be found to have significant structural weaknesses. Specific concerns include the likely lack of confinement reinforcement in columns, joints, and potential beam hinge regions, which because of the increased flexibility may have increased demands compared to earlier construction.

The trend toward lighter and more flexible construction was particularly apparent in the case of flat slab/flat plate buildings, where the use of the flat plate configuration became more common for office and residential construction up to substantial heights. Many of these buildings had neither drop panels nor column capitals, relying solely on the frame action of the floor slab and columns for resistance to lateral loads. The small shear perimeters around the columns, which are forced to transfer the gravity load shears as well as the unbalanced moment due to lateral load, can be the weak points of these structures. Post-tensioning of these slabs became common by 1960.

On the positive side, seismic code provisions were beginning to be developed, and many of the issues still being addressed today had been identified. The appearance in the codes of lateral load provisions, for both wind and earthquake, was leading to the inclusion of identified portions of the building assigned to the lateral-force-resisting system.

A number of new concepts and construction methods were coming into use. Prestressing—both pretensioned and post-tensioned—was becoming a factor in building construction. Accompanying pretensioned concrete was a greater degree of precasting, but not all precast concrete was prestressed. Precasting was done both in off-site fabricating plants and on-site. On-site precasting was most commonly associated with tilt-up construction—used mainly for low-rise commercial light industrial, and warehouse buildings—or with lift-slab construction.

Bonded post-tensioning, in both cast-in-place and precast construction, was used mainly for heavy construction such as parking garages. Adequate grouting of the tendon ducts is an issue from both construction and current condition standpoints. Adequate ductility is an issue from the seismic analysis standpoint, as it is for all prestressed construction. Prestressing cable is not ductile.

Because of the lack of service experience (with the corollary of lack of building code guidance), and novel features, many of the early structures employing the new systems had problems. Notable examples were lack of proper accommodation of length changes in prestressed systems due to continuing creep, and consequent difficulty with connections between precast elements. Even after decades of experience, these problems are not entirely solved. For early structures, these items should always be checked for possible reduction of both vertical and lateral load capacity, and for cracked or broken connections.

Connections between precast units, and between precast units and adjacent members, are vital to the integrity of the gravity- and lateral-force-resisting systems in many applications. Examples are the connections between precast roof units, between wall panels, and between walls and roofs. One of the most notable examples of the latter is the connection between wood roofs and tilt-up walls, which have failed during earthquakes in several instances. Current code provisions prohibit the use of wood ledgers in cross-grain tension or bending, in an effort to minimize the likelihood of this type of failure.

Some unbonded post-tensioned structures were also appearing about this time. Early versions frequently lacked supplementary deformed bar reinforcement for crack control and strength enhancement at overload states, a deficiency that was reduced by improved code provisions. Early versions of these systems should be checked for this problem, and for tendon corrosion as well. Another problem deserving attention is the “lock-up” of forces from unbonded tendons with vertical concrete wall systems; this has been witnessed in numerous post-tensioned structures.

In lower seismic zones in particular, support bearing length and connections between roof and floor elements and their supports should be reviewed. The need for adequate support and ductile connections may not have been appreciated in the original designs.

Precast frame buildings began to become more common about this period as well. If the frame is proportioned and connected in such a way that hinging takes place other than at the joints, then the structure should behave much like its cast-in-place counterpart. However, if hinging takes place at connections between elements, the earthquake resistance of the structure should be reviewed very carefully with respect to brittle behavior.

The use of shear walls to resist lateral forces, as part of the basic design procedure, was formalized in this period. Shear walls had often been present in one way or another, but conscious use of rigid walls at selected location, size, and strength appears to date from this period. Earlier walls that serve a comparable function can be found as bearing walls, elevator shaft walls, and infill walls in frames.

Shear wall buildings tend to be much stiffer than frame buildings—this produces the advantage of reduction of drift and deformations, and the disadvantage of attracting higher internal loads than frame buildings. One of the most serious deficiencies occurs where shear walls do not extend all the way to the foundation. Supports for discontinuous shear walls have frequently been damaged in earthquakes.

Increased use of automobiles in this period led to a substantial increase in the number of parking garages, many of which often are of concrete construction. Several features of these structures present challenges, including the size, which invites significant dimensional changes when prestressed; unfavorable environment, which promotes deterioration; irregular framing, which invites unsymmetrical response to earthquake excitation; small story heights, which may encourage weak column and short column behavior; and problems with connections in precast systems.

1960–1970. This period represents improvement and consolidation in design, code provisions, and construction. Concerns for seismic design, and hence code requirements of seismic resistance, remained concentrated mainly in California and Washington. The *Uniform Building Code*, in use mainly in the western portions of the US, was being improved continually to deal with the seismic concerns summarized earlier in this section, as technology and research provided improved resistance in design. However, the seismic sections of the UBC were not adopted or enforced in many locations, and many important deficiencies remained to be resolved. In the remainder of the United States, building codes tended to ignore seismic issues, since it was not universally recognized at the time that many other areas were at substantial seismic risk.

A major development in concrete design in this era was the conversion of the code from allowable stress methods to strength methods. Concurrently, the concepts of assigning characteristics to a designated lateral-force-resisting system were being developed. Confinement and ductility in concrete detailing were described explicitly, though still not mandated by the codes. Improvements such as continuity in positive moment reinforcement, and joint shear provisions, made their appearance.

1970–1980. This was a period of continued development of seismic design in the western United States, but attention to seismic concerns in the eastern United States was still not extensive. The major San Fernando earthquake in 1971 resulted in additional understanding of earthquake demands and detailing requirements, and may be considered a turning point in development of ductile detailing and proportioning requirements for reinforced concrete construction in the western United States. Whereas earlier codes focused on providing strengths in structural members to resist code-specified forces, the western US codes developed during this period began to focus on aspects of proportioning and detailing to achieve overall system ductility or deformability.

In beam-column moment frame constructions, requirements emerged for transverse reinforcement in beams, columns, and joints, intended to reduce the likelihood of nonductile shear failures. Requirements that columns be stronger than beams—thereby promoting strong column-weak beam inelastic deformation modes—also appeared.

For shear wall buildings, requirements for ductile boundary elements of shear walls were incorporated in codes. These provisions include transverse reinforcement to confine concrete and restrain rebar buckling, and tension lap splices designed to sustain inelastic strain levels. Provisions to reduce the likelihood of shear failure also appeared in western US codes. For tilt-up wall buildings, improvements were made in tying together the various components.

1980–Present. This period represents a continuation of improvement and consolidation in design, code provisions, and construction, as an extension of the previous period. A significant change, however, has been the broadening of attention to seismic effects, from a regional outlook to a national outlook. The *NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings* (BSSC, 1995) have become influential in FEMA efforts to focus attention on earthquakes as a national, not a regional, issue. The *Provisions* have been incorporated, with minor modifications, into the building codes in those portions of the United States not using the UBC. Since the *Provisions* differ little in their effect from the UBC, for the first time in the early 1990s there were well-established seismic code provisions in effect throughout the United States. The level of earthquake resistance of new construction should continue to improve, and there are reference standards to evaluate the capabilities of existing structures. A number of smaller magnitude earthquakes in the eastern United States and Canada demonstrated the vulnerability of the entire United States to seismic behavior, and prompted many municipalities to add appropriate design requirements.

Causes for Collapses in Reinforced Concrete Buildings. This section presents a brief discussion on causes of collapse in reinforced concrete (RC) buildings. The emphasis is on collapse as opposed to local failures. For example, the failure of a coupling beam may be dramatic, but it would not normally lead to an overall building collapse. Most collapses are ultimately caused by the deterioration and eventual failure of the gravity-load-carrying system for the structure.

- Poor Conceptual Design

  Certain structural design concepts that work well in nonseismic areas perform poorly when subjected to earthquake motions. Examples are frame structures with strong beams and weak columns, or frame structures employing soft (and weak) first stories. For either case, a single story sway mechanism can develop under lateral loading. Inelastic deformations will concentrate in this story, with the remainder of the structure staying in the elastic range of response. Even well-detailed columns will lose strength, stiffness, and energy absorbtion capacity due to the concentrated inelastic demands placed on this single story. Thus, complete structural collapse is a likely result.

Poor layout of structural walls during the initial design of a building leads to significant plan eccentricities between the center of mass and the center of lateral load resistance. Under lateral loading, torsional response modes will dominate, and large displacement demands will be placed on vertical elements farthest away from the center of stiffness. The vertical elements farthest from the center of resistance are usually perimeter columns. The large cyclic motions would typically put biaxial displacement demands on the columns; even well-detailed columns will typically fail under such extreme loading conditions.

Another poor design concept is to not provide adequate spacing between adjacent structures. When there is not adequate spacing, the buildings will "pound" against each other as they respond to the earthquake excitation. Clearly, structures are not normally designed to absorb pounding loads from adjacent structures. Also, these impulsive pounding forces can significantly alter the dynamic response of the structure in question. The 1985 Mexico City earthquake offered several examples of significant pounding damage and partial collapses of buildings due to pounding from an adjacent structure (Bertero, 1987).

- Column Failures

  Columns are the primary gravity-load-carrying members for most concrete structures. Therefore, most dramatic collapses of reinforced concrete structures during past earthquakes have been due to column failures. Common causes of column failure are discussed below.

  – Inadequate Shear Capacity

    Typical gravity and wind load designs will normally result in a design shear force significantly lower than the shear force that could be developed in a column during seismic loading. Early seismic designs that used factored loads—as opposed to a mechanism analysis—may also lead to column design shear forces well below potential shears that could act in the column during an earthquake. Another common problem is to artificially "shorten" a column by adding partial-height nonstructural partition walls that restrict the movement of the columns. The resulting short columns are stiff and attract much higher shear forces than they were designed to carry. There are numerous examples of column shear failures during past earthquakes.

  – Inadequate Confinement of Column Core

    Although most frame structures are designed using the strong column-weak beam philosophy, first-story columns often form plastic hinges during strong seismic loading. As in beam plastic hinging regions, the concrete core in a column plastic hinging region must be adequately confined to prevent deterioration of the shear and flexural strength of the column. This confinement requirement in a column is more severe because of the high axial load and shear that typically needs to be carried through the plastic hinging region. Again, there are numerous examples of failure of poorly confined columns during past earthquakes.

  – Combined Load Effects

    Poor design concepts, such as terminating shear walls above the foundation level, may result in columns that are required to carry very high axial compression and shear forces. If such columns do not have adequate confinement, there can be an explosive shear failure that is similar to the failure of the compression zone of an overreinforced beam subjected to bending and shear. A typical example would be a shear wall boundary column that extends down to the foundation while the wall terminates at the first-story level.

  – Biaxial Loading

    The problems of shear strength and confinement are commonly more severe in corner columns, especially if the building has significant eccentricity between the center of mass and the center of resistance. Corner columns need to have a higher degree of confinement (toughness) if they are to survive the biaxial displacement demands that will likely be placed on them. Examples of failure of corner columns are common in past earthquakes.

- Failures of Beams and Beam-Column Connections

Failures in beams and beam-to-column connections are most commonly related to inadequate use of transverse reinforcement for shear strength and confinement. These are typically local failures and will not necessarily lead to collapse of the building.

During severe seismic loading of a frame structure, plastic flexural hinging regions will develop at the beam ends. The shear in the beam at the formation of these hinging zones could be significantly higher than the shear forces the beam was designed for, leading to a shear failure. However, a more common problem is inadequate transverse confinement reinforcement in the beam plastic hinging zones. As the plastic hinge "works" during the earthquake, the lack of adequate confinement reinforcement will result in a steady deterioration of the shear strength and stiffness in the hinging zone.

Both beam-to-column and slab-to-column connections can suffer a significant loss of stiffness due to inadequate shear strength and anchorage capacity in the connection. Both of these "failures" are related to inadequate use of confinement reinforcement in the connection, and improper detailing of the main reinforcement anchored in or passing through the connection. For buildings on firm soil, the loss of stiffness may lead to a reduction in the displacement response—or at least very little increase—because the period of the structure tends to lengthen. However, for structures on soft soils this loss of stiffness and lengthening of the building natural period may lead to an increase in the displacement response of the structure. The increased displacements mean higher eccentric (P-Δ) loads on the structure and can cause a total collapse. The 1985 Mexico City earthquake gives some examples of this type of failure (Meli, 1987).

- Failures of Slabs at Slab-Column Connections

Slab-to-column connections that are adequate for gravity loading may suffer a punching shear failure when required to transfer gravity loads plus moments due to seismic lateral loads. Laboratory experiments as well as post-earthquake investigations have indicated that when the gravity load shear stresses are high on the critical slab section surrounding the connection, the connection has little ability to transfer moments due to lateral loads, and will fail in a brittle manner if the lateral load moments cause yielding of the slab reinforcement. This potential punching problem is a primary reason for not allowing slab-column frame structures in high seismic zones. Although punching may be considered as a "local" collapse, a potential exists for a progressive collapse of the entire structure. Some failures during the 1985 Mexico City earthquake are examples of this type of building collapse (Meli, 1987).

- Failures of Structural Walls

Structural walls with inadequately sized or poorly confined boundary elements have suffered shear-compression failures at their bases when subjected to lateral forces large enough to force the formation of a plastic hinge at the base of the wall. Again, this is typically a local failure and will not normally result in the collapse of a building, because in most structures there are either other wall elements or frame members capable of carry the gravity loads. However, such wall failures can seriously compromise the safety of the structure and make required repairs difficult to accomplish after an earthquake.

In long structural walls with a low percentage of vertical reinforcement, the tensile strains may become very large if the wall is forced to respond inelastically during an earthquake. The high tensile strains and high range of cyclic strain can lead to low-cycle fatigue fracture of the reinforcing bars. One example of this type of failure was observed following the 1985 earthquake in Chile (Wood et al., 1987). The building was a total loss and was demolished shortly after the earthquake.

- Special Problems with Precast Concrete Construction

The major issue for precast concrete construction is proper connections between the various components of the structure in order to establish a load path from the floor masses to the foundation. There are numerous examples of failures of precast buildings and tilt-up construction during earthquakes, due to inadequate connections between the different components of the structure. In many cases the components were simply not adequately connected. The true seismic demand required to be transmitted through a connection was not properly investigated, resulting in an inadequate connection.

Diaphragm flexibility and the transfer of diaphragm forces to lateral-load-resisting elements were two major problems with precast parking structures that suffered partial or total collapse during the January 1994 Northridge earthquake. Large diaphragms composed of precast elements and a thin concrete topping will deform inelastically during earthquake excitation, and the effect of these deformations on connections to the supporting elements, as well as the response of the supporting element, must be considered. Also, reinforcement in shear transfer zones between diaphragms and lateral-load-resisting elements must be carefully designed to transfer forces between these elements, considering all possible failure modes.

C6.3 Material Properties and Condition Assessment

C6.3.1 General

Each structural element in an existing building is composed of a material capable of resisting and transferring applied loads to foundation systems. One material group historically used in building construction is concrete, which includes both unreinforced and conventionally reinforced, and prestressed forms of construction. Of these, conventionally reinforced concrete has received the greatest use in buildings, from single elements such as the foundation system through primary use in frames and the superstructure. Concrete structural elements in the US building inventory have a wide diversity in size, shape, age, function, material properties, and condition, as cited in Chapter 4 of the *Guidelines*. Each of these factors has a potentially significant influence on the seismic performance of a particular building. This section is concerned with the influence of material properties and physical condition on the structural performance.

It is essential that the seismic rehabilitation effort include provisions to quantify material properties and condition during the early stages of work. Many references exist to support the determination of properties and assessment of physical condition. These references, and their recommended implementation, are addressed in this section. The focus of the materials testing and condition assessment program shall be primary gravity- and lateral-force-resisting elements.

C6.3.2 Properties of In-Place Materials and Components

C6.3.2.1 Material Properties

The primary properties of interest in an existing concrete structure are those that influence the structural analysis and rehabilitation effort. Both classical structural design and analysis of concrete, as well as typical code-prescribed requirements, are commonly based on the following strengths, which also dictate virtually all concrete component elastic and inelastic limit states:

- Compressive strength, modulus of elasticity, and unit weight of concrete; splitting tensile strength of lightweight aggregate concrete
- Yield strength and modulus of elasticity of reinforcing and connector steel
- Tensile (ultimate) and yield strength of prestressing steel reinforcement

Other material properties—such as concrete tensile and flexural strength, dynamic modulus of elasticity, and modulus of rupture; reinforcing steel bond strength and ductility; and relaxation properties of prestressing steels—may also be desirable. There are standard tests to measure these properties; most of these tests have been standardized by the ASTM. In general, accurate determination of these properties requires removal of samples of specific dimensions for laboratory testing. As indicated in Section C6.3.2.3, approximation of concrete compressive strength may also be obtained nondestructively. Samples removed shall also be examined for condition prior to mechanical testing (see Section C6.3.3).

Many factors affect the in-place compressive strength of concrete, including original constituents and mix design, age, thermal and environmental exposure history, load history, creep effects, and many others. These factors commonly introduce a certain amount of strength variability, even within specific components of a building. Additional variability may be introduced during the sampling and testing of the concrete. Thus, the derivation of existing concrete strength must be carefully approached by the design professional.

The yield strength of conventional reinforcing steel and connector materials used in concrete construction generally remains constant for the life of the building. Certain environmental conditions may weaken the steel, but these are generally confined to exposures in specific industrial and chemical plants, or buildings exposed to ocean spray or road salts. In addition, it is common for the same grade of steel (e.g., yield strength of 60,000 psi) to be used throughout a building.

The ultimate strength of prestressing steels is also generally a constant throughout the lifespan of a building. However, certain corrosive environments may alter the metallurgical structure of the steel, resulting in a weakening effect or embrittlement. In addition, relaxation of the steel, concrete volume changes, creep, and other factors may contribute to a loss of the originally introduced prestress.

Determination of other material properties may be warranted under special conditions (e.g., presence of archaic reinforcing, significant environmental exposure, special prestressing system). The design professional should consult with a concrete consultant to identify these properties if such special conditions exist.

C6.3.2.2 Component Properties

Concrete component properties include those that affect structural performance, such as physical size and thickness, geometric properties, condition and presence of degradation, and location and detailing of the reinforcing steel system. The need for tolerances in concrete construction, and factors such as concrete volume change and permeability, also affect as-built component properties. Design professionals responsible for the reanalysis of an existing building require an understanding of actual properties in order to model behavior properly.

The following component properties are cited in the *Guidelines* as important to evaluating component behavior; explanations are provided in parentheses:

- Original and current cross-sectional area, section moduli, moments of inertia, and torsional properties at critical sections (needed to establish appropriate section properties for capacity and allowable deformation checks)
- As-built configuration and physical condition of primary component end connections, and intermediate connections such as those between diaphragms and supporting beams/girders (needed to assess load transfer in the building)
- Size, anchorage, and thickness of other connector materials, including metallic anchor bolts, embedments, bracing components, and stiffening materials, commonly used in precast and tilt-up construction (materials commonly identified as “weak links” in building performance)
- Characteristics that may influence the continuity, moment-rotation, or energy dissipation and load transfer behavior of connections (needed to assess load transfer, and to understand connection behavior and implications on building deformation)
- Confirmation of load transfer capability at component-to-element connections, and overall element/structure behavior (needed to ensure element integrity and stability)

An important starting point for developing component properties is the retrieval of original design/construction records, including drawings. Such records may then be used at the building site for as-built comparison and conformance checks. The process of developing component properties and inspecting of the physical condition of a concrete structure is commonly referred to as “condition assessment” or “condition survey.”

C6.3.2.3 Test Methods to Quantify Properties

Concrete. The sampling of concrete from existing structures to determine mechanical and physical properties has traditionally employed the use of *ASTM C 823, Standard Practice for Examination and Sampling of Hardened Concrete in Constructions* (ASTM, 1995). All sampling shall be preceded by nondestructive location of underlying reinforcing steel to minimize sampling effects on the existing structure. In general, the property of greatest interest is the expected compressive strength, f'_c .

The accurate determination of mechanical properties of existing concrete in a building requires the removal of core samples (sawed beams for flexural tests) and performance of laboratory testing. The sampling effort shall follow the requirements of *ASTM C 42, Method of Obtaining and Testing Drilled Cores and Sawed Beams of Concrete* (ASTM, 1990) (sawed beams should not be used unless core extraction is prohibitive). The testing of core concrete to determine mechanical properties shall follow specific ASTM procedures relative to the property of interest:

C 39, Standard Test Method for the Compressive Strength of Cylindrical Concrete Specimens

C 496, Test of Splitting Tensile Strength of Concrete

C 78, Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)

C 293, Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Center-Point Loading)

Derivation of in-place concrete strength from core samples taken requires statistical analysis and correlation of core strength to actual strength. A recently developed procedure (Bartlett and MacGregor, 1995) for this correlation involves the following equation:

$$f^{i}_{c,ip} = F_{l/d}F_{dia}F_rF_{mc}F_df_c \tag{C6-1}$$

where: $f^{\,i}_{c,ip}$ is the equivalent in-place strength for the *ith* core sample taken from a particular concrete class, and f_c is the measured core strength. The other expressions are strength correction factors for the effect of length to diameter ratio ($F_{l/d}$), diameter of the core (F_{dia}), presence of reinforcing steel (F_r), moisture condition of the core (F_{mc}), and strength loss due to damage during drilling (F_d). Mean values for these coefficients may be used, as derived from the following table:

Factor	Mean Value	Variability (%)
$F_{l/d}$: l/d ratio[a]		
Soaked[b]	$1 - \{0.117 - 4.3 \times (10^{-4}) f_c\} \times (2 - l/d)^2$	$2.5(2 - l/d)^2$
Air dried[b]	$1 - \{0.144 - 4.3 \times (10^{-4}) f_c\} \times (2 - l/d)^2$	$2.5(2 - l/d)^2$
F_{dia}: Core diameter		
50 mm	1.06	11.8
100 mm	1.00	0.0
150 mm	0.98	1.8
F_r: bars present		
None	1.00	0.0
One bar	1.08	2.8
Two bars	1.13	2.8
F_{mc}: Core moisture		
Soaked[b]	1.09	2.5
Air dried[b]	0.96	2.5
F_d: Damage due to drilling	1.06	2.5

[a] f_c is in MPa; for f_c in psi, the constant is $-3(10^{-6})$.

[b] Standard treatment specified in ASTM C 42.

This procedure should be utilized for determining the compressive strength for use in structural calculations, using the following approach. The equivalent in-place concrete strength for structural analysis shall consist of the mean of the converted core strengths from Equation C6-1 as:

$$f_{c,ip} = \frac{(f^{1}_{c,ip} + f^{2}_{c,ip} + \ldots + f^{n}_{c,ip})}{n} \tag{C6-2}$$

where $f^{1}_{c,ip}, f^{2}_{c,ip}, \ldots f^{n}_{c,ip}$ are the equivalent compressive strengths computed from individual cores sampled (as computed via Equation C6-1) and n is the total number of cores taken from the particular concrete class.

The variability in measured core strengths should also be checked to: (1) determine the overall quality of the concrete, (2) determine if enough core samples were removed, (3) eliminate error, (4) properly identify outliers, and (5) make any needed adjustments to $f_{c,ip}$. The standard deviation, variance, and coefficient of variation should be checked via the following equations:

$$Q_c = [(f^{1}_{c,ip} - f_{c,ip})^2 + (f^{2}_{c,ip} - f_{c,ip})^2 + \ldots + (f^{n}_{c,ip} - f_{c,ip})^2] \tag{C6-3}$$

$$S_c = (Q_c)^{0.5} \tag{C6-4}$$

$$C.O.V. = \left[\frac{S_c}{f_{c,ip}}\right] \qquad \text{(C6-5)}$$

where:

Q_c = Variance

S_c = Standard deviation

C.O.V. = Coefficient of variation

Further reduction of the equivalent strength values is suggested by the literature (Bartlett and MacGregor, 1995) to improve upon the confidence in results; it is reported that the probability that the in-place compressive strength is less than f'_c is 13.5% (rounded to 14%). As opposed to further reduction of correlated values, if the *C.O.V* is less than 14%, then the mean strength from testing may be used as the expected strength in structural analyses ($f'_c = f_{c,ip}$). The *C.O.V.* cut-off value was established to account for testing errors, damage from improper coring, and other factors that may alter individual test results as noted in the literature. However, if the coefficient of variation from this testing exceeds 14% or the results are greater than 500 psi below specified design, f'_c , further assessment of the cause through additional sampling/testing is needed. Such causes might be, among others, poor concrete quality, an insufficient number of samples/tests, or sampling or testing problems. In general, the expected strength taken from results with higher variation should be a maximum of the mean less one standard deviation ($f'_c \le f_{c,ip} - S_c$). The design professional may further reduce the expected strength (and gain confidence in actual strength levels) if concrete quality or degradation are observed. The results should also be examined to ensure that one or more outliers (e.g., individual test results with large differences from other tests) are not influencing results. Outliers should be dispositioned per *ASTM E 178, Standard Practice for Dealing with Outlying Observations*.

Appropriate values for other strengths (e.g., tensile, flexural) shall be derived from the referenced ASTM tests and accepted statistical methods.

Other nondestructive and semi-destructive methods have been established to estimate the in-place compressive strength of concrete (ACI, 1995a). Methods applicable to hardened concrete, with referenced ASTM procedures, include the ultrasonic pulse velocity method (*ASTM C 597*), penetration resistance methods (*ASTM C 803*), and surface hardness or rebound methods (*ASTM C 805*). However, to date, these methods have demonstrated limited correlation to strength, with high internal coefficients of variation. Because of these constraints, and the need for calibration standards for each method, substitution of these methods for core sampling and laboratory testing is prohibited. These methods may be economically used, however, to qualitatively check concrete strength uniformity throughout the structural system as opposed to core drilling samples. The guidance of ACI Report 228.1R-95 (ACI, 1995) should be used if nondestructive methods are to be employed in this manner.

Conventional Reinforcing Steel. The sampling of reinforcing and connector steels shall be done with care and in locations of reduced stress; sampled areas should be repaired unless an analysis indicates that the local damage produced is acceptable. Sample sizes should be per *ASTM A 470, Standard Test Methods and Definitions for Mechanical Testing of Steel Products*, with longitudinal, planar, or stirrup bars used as opposed to ties. There shall be a maximum of one sample taken at any one cross-section location, and samples should be separated by at least one development length (*ACI 364.1R*).

Determination of tensile and bend strength and modulus of elasticity of conventional reinforcing and connector steels shall be as defined in *ASTM A 370*. Included in the determination of reinforcing steel strength properties is the characterization of material type; bond strength with the existing concrete may also be of interest, but this is extremely difficult to accurately measure in field conditions. Reinforcing steels used before 1950 had various cross-sectional shapes (e.g. square, rectangular, round), surface conditions (e.g., ribbed, deformed, smooth, corrugated), and proprietary additions (e.g., herringbone shape, special deformations). Each of these characteristics may contribute to overall performance of the particular structure. The history of reinforcing steel and mechanical properties is summarized in *Evaluation of Reinforcing Steel Systems in Old Reinforced Concrete Structures* (CRSI, 1981). This document also recommends that older reinforcing steel systems be treated as 50% effective, the primary problems being with tensile lap splice deficiencies.

Connector steel properties shall be determined either via sampling and laboratory testing using *ASTM A 370*, or by in-place static tensile testing following the provisions of *ASTM E 488*, *Standard Test Methods for Strength of Anchors in Concrete and Masonry Elements.*

Prestressing Steel. Similar to conventional reinforcing, the yield and tensile strengths and modulus of elasticity of prestressing steels may be derived from testing in accordance with *ASTM A 370*. A maximum of one tendon per component shall be sampled, with a replacement tendon installed.

C6.3.2.4 Minimum Number of Tests

Determination of mechanical properties for use in the reanalysis of an existing building involves the completion of physical tests on *primary* component materials. Testing is not required on secondary components and other nonstructural elements, but may be performed to better analyze the building at the discretion of the design professional. The number of tests needed depends on many factors, including the type and age of construction, building size, accessibility, presence of degradation, desired accuracy, and cost. In particular, the costs for obtaining a statistically robust sample size and completing the destructive tests with a high level of confidence may be significant. A minimum level of testing for key properties that account for building size, concrete structure type, different classes of concrete, and variability was identified in *Guidelines* Section 6.3.2.4. It is recommended that a more comprehensive sampling program be established.

Minimum Sample Size. The minimum number of tests for determining material properties was identified from references including *ACI 228.1R* (concrete), various ASTM publications, and CRSI (reinforcing steel) guidelines. Typical coefficients of variation in concrete and steel materials were also cited from these references. In general, there is a statistical relationship between the minimum test quantity and the accuracy of the derived property. If prior information (e.g., design/construction records) exists, significantly higher confidence in the property of interest will be obtained with a reduced number of tests. Recent research (Bartlett and MacGregor, 1995) has shown that a minimum of three test sample should be taken if error is to be avoided, but at least six samples should be detected to identify outliers or specific values that deviate greatly from the others. Other documents (e.g., *ACI 228.1R*) have suggested that at least 12 cores be taken and tested to assess strength. The number of tests prescribed in the *Guidelines* was established with these reports as a basis. For small residential buildings, it is considered practical to obtain the expected strength from a small number of samples (such as three) as long as the coefficient of variation (*C.O.V.*) is low. However, with a larger tall building the number of tests may well exceed the minimum.

For reinforcing and prestressing steels, the minimum sample size is smaller than for concrete, because of material homogeneity, lower property variability, common material grades typically used throughout buildings, damage caused by sampling and need for repair, and ability to use samples to derive multiple properties. The sample size for prestressing steel shall be based on design information. If these data do not exist, sampling and testing are required. Because of the prestress, extreme care must be taken during disassembly.

Increased Sample Size. A higher degree of accuracy in material properties may be acquired by increasing the number of tests performed, supplementing required sampling/laboratory testing with rapid nondestructive methods, or using Bayesian statistics to gain further confidence.

Conventional statistical methods, such as those presented in *ASTM E 122* may also be used to determine the number of tests needed to achieve a specific confidence level. In general, these practices typically lead to a sample size much larger than the minimum number prescribed in the *Guidelines*. For reasons including access restrictions and cost, the design professional should consider using *ASTM E 122* or similar references to establish the actual sample sizes for a particular building.

Several nondestructive methods, including ultrasonic pulse velocity testing, may be effectively used to estimate concrete compressive strength and other in situ properties. Calibration of these methods with core test results is necessary for desired accuracy. The results may be used to improve confidence in representation of the core test results.

Bayesian statistics provide a means for improving confidence in material properties derived from a sample when prior information is available (e.g., design drawings, construction test records). A combination of strength data from cores and nondestructive methods may also be systematically combined via Bayes' theorem to obtain mean and standard deviation of compressive strength. This approach may also be used to justify use of a smaller sample size (e.g., minimum number of tests), especially if prior knowledge exists and a single concrete class was used in construction. Further information on the use of Bayesian statistics in material property selection is contained in Kriviak and Scanlon (1987) and Bartlett and Sexsmith (1991).

C6.3.2.5 Default Properties

Default values for key concrete and reinforcing steel mechanical properties were identified from the literature (e.g., CRSI, 1981; Merriman, 1911) in the Section 6.2 tables. Default values are provided for situations in which the design professional does not have materials test data from which in-place strengths may be derived. While these values have been further reduced in *Guidelines* Section 6.3.2.5, the design professional is cautioned against their use, as lower-strength or poorer quality materials may exist in the specific building in question. Concrete compressive strength in particular may be highly variable, even within a specific building. It is highly recommended that at least the minimum amount of testing in *Guidelines* Section 6.3.2.4 be carried out for confirmation of properties.

Another common condition in historic concrete construction was the use of contractor-specific proprietary systems, including floors and decks. Material properties in these proprietary designs may have been published in trade publications or other texts. The design professional is encouraged to research such references if the use of a proprietary system in the building is identified. Use of default values for these proprietary systems is not recommended. Also, as noted in CRSI (1981), it is recommended that a 50% reduction in effectiveness be applied to the reinforcing steel systems in historic construction.

C6.3.3 Condition Assessment

C6.3.3.1 General

The scope of the condition assessment effort—including visual inspection, component property determination, and use of supplemental testing—shall be developed by the design professional. The recommended scope of work includes all primary vertical- and lateral-load-resisting elements and their connections. Procedures for conducting the assessment and methods for use in assessing physical condition are referenced in the following section.

C6.3.3.2 Scope and Procedures

A condition assessment following the recommended guidelines of *ACI 201.2R* is recommended to be performed on all primary and secondary concrete elements of a building. The following steps should be considered.

1. Retrieve building drawings, specifications, improvement or alteration records, original test reports, and similar information.

2. Define the age of the building (e.g., when the building materials were procured and erected).

3. Compare age and drawing information to reference standards and practices of the period.

4. Conduct field material identification via visual inspection and in-place nondestructive testing of concrete.

5. Obtain representative samples from components and perform laboratory tests (e.g., compression, tensile, chemical) to establish in-place material properties per *Guidelines* Section 6.3.2.3. Samples shall be taken at random throughout the concrete building and elements. Test methods identified in Section 6.3.2 shall be used.

6. Determine chloride content and depth profile in concrete, if reinforcing steel corrosion is suspected, and determine the amount of loss of reinforcement due to corrosion, where applicable.

7. Visually inspect components and connections of the structural system to verify the physical condition.

Further information regarding the condition assessment of concrete structures may be found in *ACI 364.1R-94, Guide for Evaluation of Concrete Structures Prior to Rehabilitation*, and *ACI 201.2R-92, Guide for Making a Condition Survey of Concrete in Service*.

The samples removed for material property quantification may also be used for condition assessment. Significant data relative to the condition and quality of concrete (through petrographics and other tests) and reinforcing steel (degree of corrosion) may be established. In the event that degradation is observed in the visual assessment or review of retrieved samples, additional nondestructive and destructive tests should be used to quantify the extent. Such testing, referenced in the following paragraphs, should be performed by qualified personnel and testing firms.

Supplemental Test Methods for Concrete. Numerous nondestructive and destructive test methods have been developed for the examination and mapping of degradation and damage in concrete structures. Nondestructive methods (NDE) that may be used and their capabilities include:

Method	Capability/Use
Ultrasonic pulse-echo and pulse velocity	Indication of strength, uniformity, and quality; presence of internal damage and location; density and thickness estimation; location of reinforcing.
Impact-echo	Presence and location of cracking, voids, and other internal degradation.
Acoustic tomography	Presence and accurate location of cracking, voids, and other internal degradation.
Infrared thermography	Detection of shallow internal degradation and construction defects, delaminations, and voids.
Penetrating radar	Same as thermography; greater depth of inspectability.
Acoustic emission	Real-time monitoring of concrete degradation growth and structural performance.
Radiography	Location, size, and condition of reinforcing steel, and internal voids and density of concrete.
Chain-drag testing	Presence of near-surface delaminations and other degradation.
Crack mapping	Surface mapping of cracks to determine source, dimensions, activity level, and influence on performance.
Surface methods	Estimation of compressive strength and near-surface quality (methods such as Windsor probe, rebound hammer).

The practical application and usefulness of these methods is defined in numerous ACI and ASCE publications, including *ASCE Standard 11-90*, which compares and contrasts method capabilities for concrete element and damage types.

Additional physical properties for concrete may also be determined through use of other laboratory tests. Petrography (*ASTM C 856*) includes a series of laboratory tests performed on samples to assess concrete condition. These properties include entrained air quantity, depth of carbonation, degree of hydration, aggregates used, unit weight estimate, permeability, cement-aggregate reaction, and others.

Reinforcing System Assessment. The configuration and condition of reinforcing steel (conventional or prestressed) is especially critical to the future performance of the lateral- and vertical-force-resisting structural elements. The reinforcing steel is necessary to perform a variety of load resistance and transfer functions; to provide suitable ductility to the component and its connections; to prevent excessive straining, tensile stress development, and cracking in concrete from occurring; and for other purposes. Several means of evaluating the existing reinforcing steel system exist, including:

- Removal of cover concrete and direct visual inspection
- Local core sampling through a reinforcing bar(s)
- Nondestructive inspection using electromagnetic, electrochemical, radiographic, and other methods

Each method has positive and negative aspects. The greatest assurance of conventional or prestressed steel condition and configuration is gained through exposure and inspection. Critical parameters such as lap splice length, presence of hooks, development with concrete, and degree of corrosion can all be addressed in this manner. Of particular value is the ability to assess existing reinforcing detailing at critical component connections (for comparison to drawings and current code provisions). However, the expense, damage, and debris generated by this effort may be significant and disruptive to building use. The design professional should consider exposing a percentage of connections and the local reinforcing steel system to confirm drawing details and integrity of construction per the *Guidelines*.

Local core sampling through reinforcing steel is generally not a recommended practice because of the damage caused to the particular bar. However, during removal of cores for concrete strength testing, a sample containing portions of a bar may be inadvertently obtained. Such samples often allow direct visual inspection of local bar condition and interaction with surrounding concrete, and this information should be recorded.

Improvements in the area of nondestructive testing continue to be made. Existing proven technologies to identify bar location and approximate size include electromagnetic methods (via pachometers, profometers, and similar equipment), radiography, penetrating radar, and infrared thermography. To assess the activity level of corrosion in conventional reinforcing steel, half-cell potential (*ASTM C 876*), electrochemical impedance, and electrical resistivity methods have been used with some success. Electromagnetic methods have enjoyed the most use and have a good accuracy for round cross-section bars in uncongested areas (e.g., outer longitudinal steel in component spans). Reduced accuracy is demonstrated for locating square and other bar shapes, and at connections. Radiography, radar, and thermography have specific applications for which they provide important bar location information; however, available equipment capability, geometry, bar congestion, and component thickness present limitations to practical application.

To obtain details of prestressing steel location, remaining prestress, and physical condition requires direct exposure and inspection of anchorages, ducts (unbonded), and tendons (bonded). Measurement of remaining prestress in unbonded systems may be physically possible, depending on the system used and the end connection configuration. For accessible unbonded tendons, measurement of remaining prestress force may occur through use of calibrated hydraulic jacks and a lift-off procedure at one anchorage point, or through magnetic methods. Several nondestructive tests, including "coring stress relief," have also been used to assess existing prestress levels (Brooks et al., 1990). Observation of corrosion in prestressing systems must also be carefully treated, as prestressing steel is susceptible to sudden fracture from hydrogen (corrosion byproduct) embrittlement, and often requires its full cross-sectional area to sustain applied loads. Widespread corrosion is indicative of a need for major rehabilitation.

Identification of the steel used in reinforcing systems may also necessitate the use of chemical testing on removed samples. The provisions of *ASTM A 751, Methods, Practices, and Definitions for Chemical Analysis of Steel Products* should be followed in this regard. If the carbon equivalent must be calculated to support welded attachment, the methodology in *AWS D1.4-92* (AWS, 1992) shall be followed.

Additional details on NDE and destructive testing are contained in *ASCE Standard 11-90* (ASCE, 1990).

Load Testing. A more thorough understanding of individual concrete components or elements may be gained through the performance of in-place load testing. Simulated gravity or lateral loads may be applied to an exposed component or element, with the response to loading measured via instrumentation (e.g., strain gauges, transducers, deflectometers) and data collection means. The measured results may be used to define structural performance under future load events and improve knowledge of condition and configuration. The aspect of performing load tests on concrete components is well defined in *ACI 437-94* and Chapter 20 of *ACI 318-95*. Load test results are also an acceptable means of establishing component capacity as stated in the model building codes (e.g., UBC), especially for elements constructed with alternative materials or techniques, and those with questionable capacity.

Limitations related to load testing include the expense of test performance, access requirements to the component(s), potential damage inflicted during the test, and difficulties posed by load application (e.g., high magnitude) and interpretation. In general, load testing has limited practicality in an existing, occupied building. However, it remains a viable option for certain components and building types.

Summary. The design professional of record is responsible for establishing the condition assessment and testing methods to be used as part of a seismic rehabilitation effort. Experienced personnel, proper equipment and procedures, accurate testing, and prudent interpretation of results are imperative to the determination of component/element structural capacity and deformation limits.

C6.3.3.3 Quantifying Results

The quantitative results from the condition assessment—such as component dimensions, significance of damage, and connection continuity—must be factored into the structural analysis and rehabilitation planning. Few resources exist that provide the design professional with assistance in quantifying the effects of damage on performance. If significant reinforcement corrosion or concrete loss is observed, it may be necessary to use load testing to assess in-place strength. If degraded elements are to be reused in the building, special attention should be given to mitigation of the degradation mechanism and stabilization of the element(s).

C6.3.4 Knowledge (κ) Factor

As noted in *Guidelines* Section 2.7.2 and the *Commentary* on it, a factor (κ) associated with the relative knowledge of as-built configuration and condition is used in component capacity and allowable deformation calculations. For concrete components, including foundations and columns, complete knowledge of reinforcing configuration and continuity is not likely to exist even if the original drawings are located. Other factors, such as actual material strength and resistance to applied loads, may not be completely understood. It is recommended that the lower κ factor of 0.75 be used if any concerns about condition or performance exist. This will provide a further factor of safety against unknown conditions.

C6.3.5 Rehabilitation Issues

After structural analysis of the building is completed, it may be determined that parts or all of the structure are seismically deficient. If rehabilitation is planned, a number of concrete materials issues must be considered in the design. Of paramount importance to concrete structure rehabilitation are the size, condition, location, and continuity of the reinforcing steel system, especially at element connections. It is recommended that the design professional pay significant attention to the reinforcing system in existing structures for reuse, attachment, treatment, and modification. If the strength, ductility, or confinement provided by the existing reinforcing system is in question, further examination of in-place conditions shall be performed. Section 6.3.6 of the *Guidelines* further addresses connection issues.

If a rehabilitation program is selected and attachment to the existing structure is required, a number of factors that may influence behavior must be addressed, including:

- Attachment to existing reinforcing steel, including required development, splicing, and mechanical or welded attachment
- Level of steady-state stress present in the components to be reinforced, and its treatment
- Elastic and strain-hardening properties of existing components and preservation of strain compatibility with any new reinforcement materials
- Confinement reinforcing steel and ductility requirements for existing and new components and their connections
- Prerequisite efforts necessary to achieve appropriate fit-up, continuity, and development
- Historic preservation issues
- Load flow and deformation at connections (especially beam-column joints, diaphragm, and shear wall connections where significant load transfer occurs)
- Treatment and rehabilitation of existing damage found during the condition assessment (e.g., concrete cracks, corrosion damage)

Many other material-related issues must be considered when planning seismic rehabilitation efforts. Increased attention should be paid to primary components and those with limited redundancy.

The design of all new components in the rehabilitation program shall be in accordance with the applicable state and local building codes and industry-accepted standards. Compatibility between new and existing components must be maintained at all times.

C6.4 General Assumptions and Requirements

C6.4.1 Modeling and Design

C6.4.1.1 General Approach

Procedures in the *Guidelines* for analysis and design of concrete components and elements are based on the analysis and design procedures of *ACI 318-95* (ACI, 1995). Those provisions govern, except where these *Guidelines* specify different procedures and where it is shown by rational analysis or experiment that alternate procedures are appropriate. Some modifications to the procedures of *ACI 318-95* are necessary because, whereas *ACI 318-95* covers new construction, these *Guidelines* cover existing construction and its seismic rehabilitation.

ACI 318-95 is a design document for new materials that includes proportioning and detailing requirements intended to produce serviceable and safe structures. Many of the rules of *ACI 318-95* are designed to automatically preclude certain types of nonductile failure modes for the design loading. An existing building structure may not have been designed according to the current requirements of *ACI 318-95*, and its design may not have considered the currently recognized seismic loading. Therefore, it is possible that seismic response may be controlled by brittle or low-ductility failure modes. The engineer is cautioned to examine all aspects of possible building response—including, but not limited to, response modes associated with flexure, axial load, shear, torsion, and anchorage and reinforcement development.

Commonly used Analysis Procedures identify design actions only at specific locations of a component, typically at sections where maximum design actions are expected. When this is the case, it is necessary to check separately that design strengths are not exceeded at other sections. Figure C6-1 illustrates how this may be done for a beam component of a beam-column moment frame analyzed by the linear procedures of Chapter 3. In Figure C6-1a, the calculated design moments at the component ends do not exceed the design moment strengths. These design beam end moments can be used, along with the known gravity load and beam geometry, to determine design moments and shears at all sections along the component length, which can then be compared with design strengths at all sections. In Figure C6-1b, the calculated design moments at the beam ends exceed the design moment strengths, indicating inelastic response of the component. To determine the internal beam actions corresponding to this loading case, the design end moments are replaced with the design moment strengths (the maximum moments that can be developed at the beam ends). With this information, statics can again be used to construct the internal shear and moment diagrams, which can in turn be compared with design strengths at all sections along the length. For the case shown, the design moment diagram lies within the design strengths, so it is assured that inelastic action occurs by flexure at the beam ends. If the design shear or moment diagram at any section exceeds the design strength at that section, then inelastic action at that section would be identified, and the design actions would have to be adjusted accordingly or the component would have to be rehabilitated to prevent inelastic action.

Inelastic response along the length of a component is most likely if there are changes in design strength along the length or if gravity load effects are relatively large. Figure C6-2 illustrates these for a beam. Because of either large gravity loading or long beam span, the maximum positive design moment occurs away from the beam end. Coupled with reductions in longitudinal reinforcement, positive plastic moment flexural hinging along the span is likely under the design earthquake plus gravity loading.

C6.4.1.2 Stiffness

Stiffness of a reinforced concrete component depends on material properties (including current condition), component dimensions, reinforcement quantities, boundary conditions, and stress levels. Each of these aspects should be considered and verified when defining effective stiffnesses.

Reinforced concrete texts and design codes prescribe precise procedures for stiffness calculation. Most of these procedures were developed from tests of simply-supported reinforced concrete flexural members, loaded to relatively low stress levels. The results often have little relation to effective stiffness of a reinforced concrete component that is interconnected with other components, and subjected to high levels of lateral load. Actual boundary conditions and stress levels may result in significantly different effective stiffnesses. Experience in component testing suggests that the variations in stiffness from one component to another are largely indeterminate. The engineer carrying out an evaluation of an existing building needs to be aware that

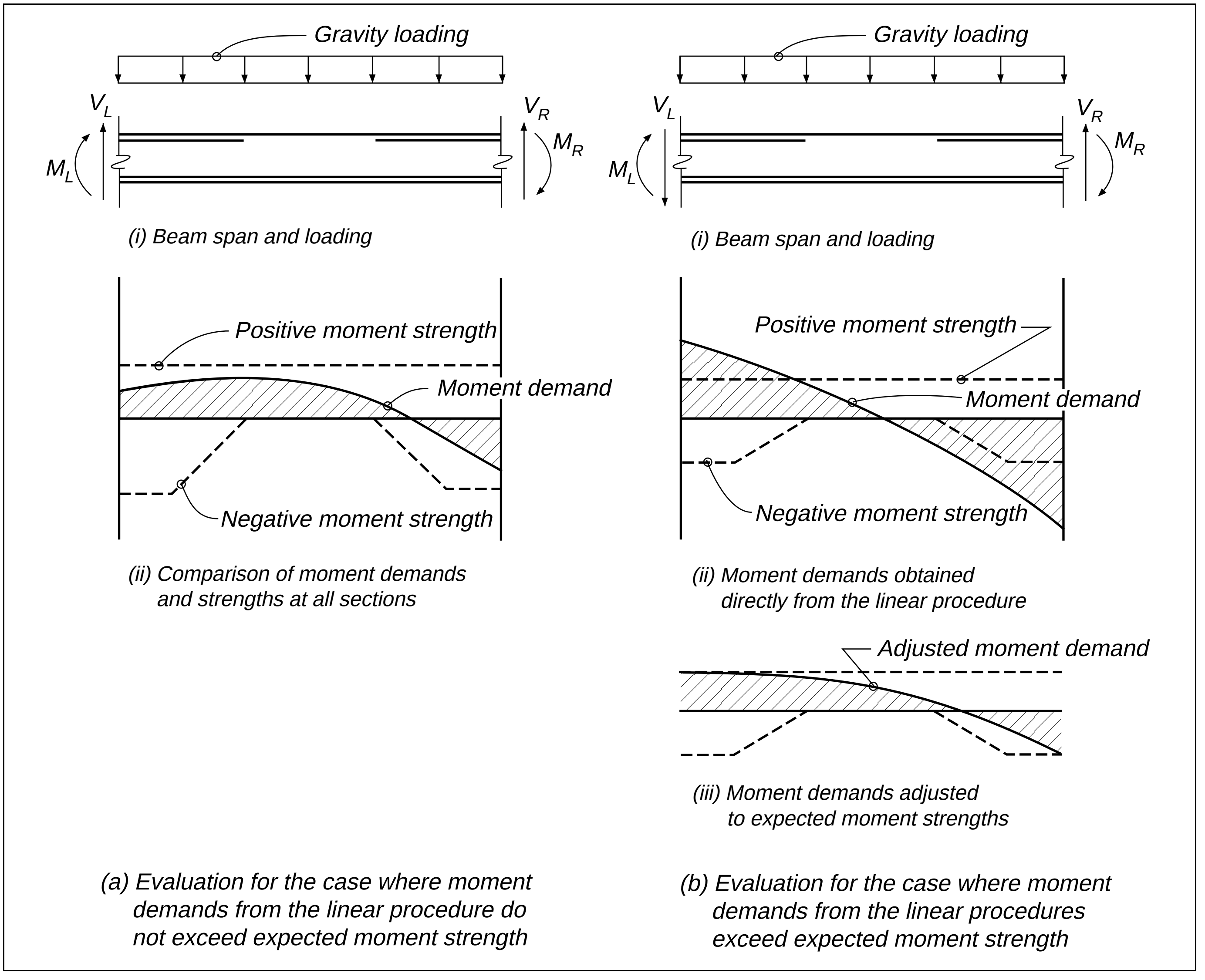


Figure C6-1 Evaluation of Beam Moment Demands of All Sections Along Span

a range of stiffnesses is possible for any set of nominal conditions, and that variations within the range may have a considerable impact on the final assessment.

The typical sources of flexibility for a relatively squat reinforced concrete cantilever wall are illustrated in Figure C6-3. These include flexure, shear, and reinforcement slip from adjacent connections (e.g., foundations, beam-column joints, walls). Flexure tends to dominate for relatively slender components (*h/l* exceeding about five). Shear and reinforcement slip tend to dominate for relatively lower aspect ratios. Whereas flexure and shear rigidities can be estimated acceptably with available mechanics procedures, the effects of reinforcement slip—which can be appreciable or even dominant—cannot be predicted accurately. For columns and shear walls subjected to appreciable axial stress variations under earthquake loading, it is important to also model axial flexibility.

A. Linear Procedures

The linear procedures of Chapter 3 were developed under the assumption that the stiffness of the analysis model approximates the stiffness of the building as it oscillates at displacement amplitudes near an effective yield condition. While this is an imprecise definition, it is clear that the target stiffness in many cases will be considerably less than the gross-section stiffness

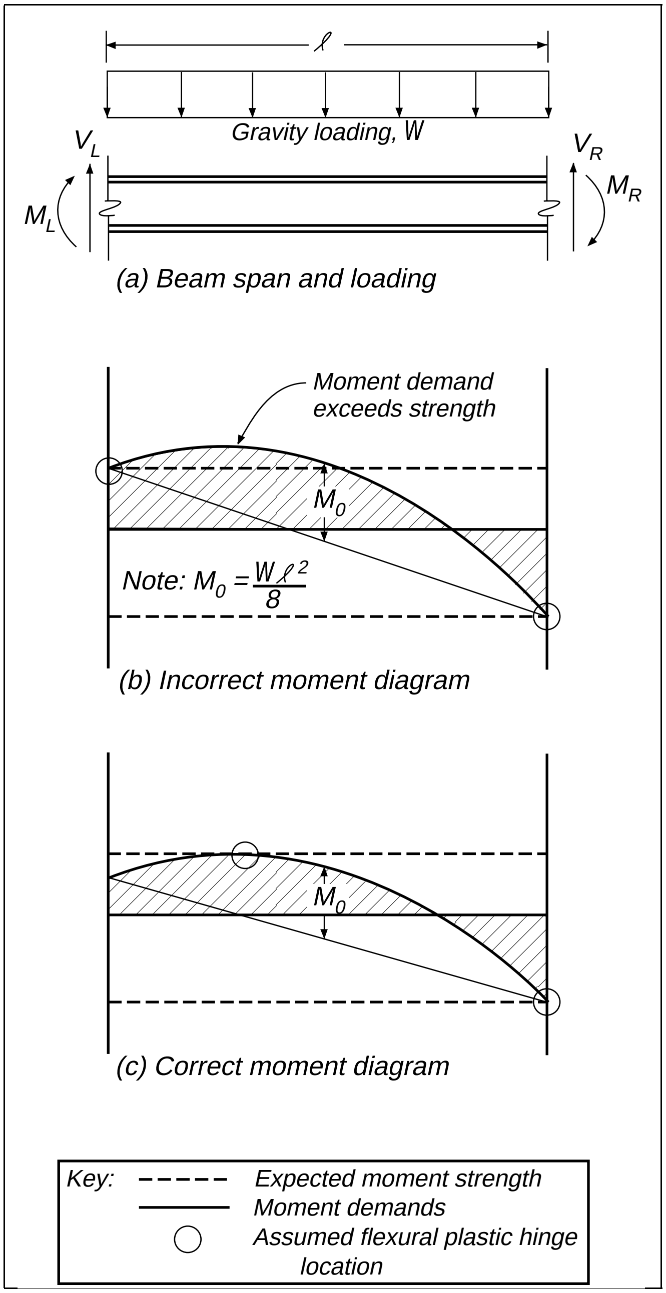


Figure C6-2 Determination of Correct Locations of Beam Flexural Plastic Hinges

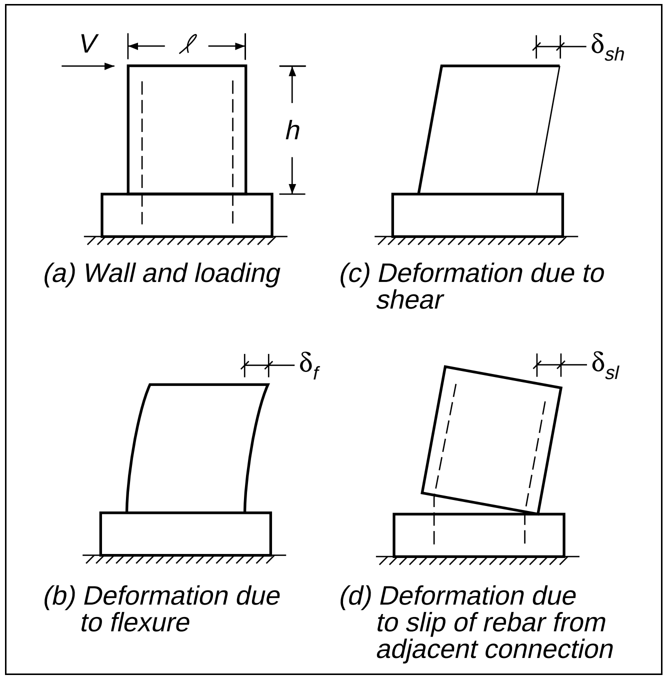


Figure C6-3 Sources of Flexibility in a Wall

commonly used in conventional design practice. The target stiffness for a given component will depend somewhat on the sources of deformation and the anticipated stress levels, as suggested by the following.

- For a **flexure dominated component**, effective stiffness can be calculated considering well-developed flexural cracking, minimal shear cracking, and partial slip of reinforcement from adjacent joints and foundation elements. Flexural stiffness can be calculated according to conventional procedures that take into consideration the variation of flexural moment and cracking along the component length. Shear stiffness may be approximated based on the gross section. Reinforcement slip (which may as much as double the overall flexibility) can be calculated by assuming appropriate stress-slip relations. Where stress levels under design load combinations are certain to be less than levels corresponding to significant cracking, uncracked flexural stiffness may be appropriate.

- For a **shear dominated component**, the onset of shear cracking commonly results in a dramatic reduction in effective stiffness, and may be considered to represent the end of elastic behavior for the component. Therefore, for shear-dominated components the effective stiffness may be based on the gross-section properties, considering flexure and shear. Stiffness reduction to account for reinforcement slip from foundation elements may be appropriate.

- For an **axial dominated component**, the appropriate stiffness depends on whether the axial load is tensile or compressive under the design load combinations.

Where it is compressive, the stiffness can be derived from the gross-section or uncracked transformed-section properties. Where it is tensile, and of sufficient magnitude to result in cracking, stiffness based on the reinforcement only should be used.

In most cases it will be impractical to calculate effective stiffnesses directly from principles of basic mechanics. Instead, the effective stiffness for the linear procedures of Chapter 3 may be based on the approximate values of Table 6-4.

Some of the stiffness values given in Table 6-4 vary with the level of axial load, where axial load is a force-controlled action including gravity and earthquake loading effects calculated according to the procedures specified in Chapter 3. In statically indeterminate structures, the calculated actions will depend on the assumed stiffness, and in certain cases it will not be possible to identify a stiffness from Table 6-4 that results in an action that is consistent with the assumed stiffness. For example, a column may be assumed to be in compression, resulting in a flexural stiffness of $0.7E_cI_g$; the analysis with this stiffness produces column tension. On the other hand, if the same column is assumed to be in tension, resulting in a flexural stiffness of $0.5E_cI_g$, the analysis indicates that the column is in compression. For this column, it is acceptable to assume an intermediate stiffness of $0.6E_cI_g$.

B. Nonlinear Procedures

The nonlinear procedures of Chapter 3 require definition of nonlinear load-deformation relations. For the NSP it is usually sufficient to define a load-deformation relation that describes behavior under monotonically increasing lateral deformation. For the NDP it is also necessary to define load-deformation rules for multiple reversed deformation cycles.

Figure C6-4 illustrates load-deformation relations that may be appropriate to the NSP of Chapter 3. Figure C6-4a is identical in content to Figure 6-1. The following aspects of these relations are important.

- **Point *A*** corresponds to the unloaded condition. The analysis must recognize that gravity loads may induce initial forces and deformations that should be accounted for in the model. Therefore, lateral loading may commence at a point other than the origin of the load-deformation relation.

- **Point *B*** has resistance equal to the nominal yield strength. Usually, this load is less than the nominal strength defined in Section 6.4.2.

- **The slope from *B* to *C***, ignoring effects of gravity loads acting through lateral displacements, is usually taken as equal to between zero and 10% of the initial slope. Strain hardening, which is observed for most reinforced concrete components, may have an important effect on redistribution of internal forces among adjacent components.

- **The ordinate at *C*** corresponds to the nominal strength defined in Section 6.4.2. In some computer codes used for structural analysis it is not possible to specify directly the value of resistance at point *C*. Rather, it is possible only to define the ordinate at *B* and the slope for loading after *B*. In such cases, results should be checked to ensure that final force levels following strain hardening are consistent with expected resistance for that deformation level. Strain hardening to values considerably in excess of the nominal strength should be avoided.

- **The drop in resistance from *C* to *D*** represents initial failure of the component. It may be associated with phenomena such as fracture of longitudinal reinforcement, spalling of concrete, or sudden shear failure following initial yield.

- **The residual resistance from *D* to *E*** may be non-zero in some cases, and may be effectively zero in others.

- **Point *E*** is a point defining the useful deformation limit. In some cases, initial failure at *C* defines the limiting deformation, in which case *E* is a point having deformation equal to that at *C* and zero resistance. In other cases, deformations beyond *C* will be permitted even though the resistance is greatly reduced or even zero-valued.

Many currently available computer programs can only directly model a simple bilinear load-deformation relation. For this reason it is acceptable for the NSP to represent the load-deformation relation by lines connecting points *A*-*B*-*C* as shown in Figure C6-4(b). Alternatively, it may be possible and desirable to use more detailed load-deformation relations such as the relation illustrated in Figure C6-4(c).

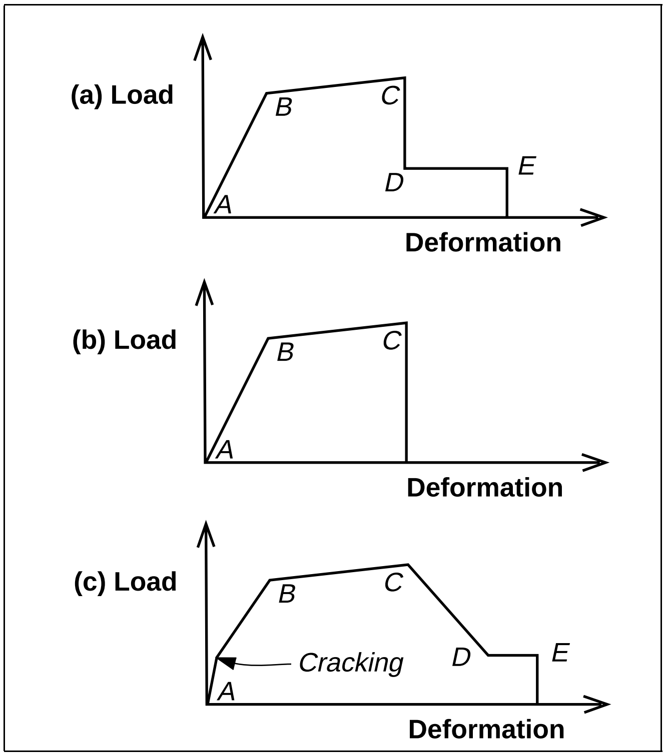


Figure C6-4 Typical Load-Deformation Relations Suitable for Nonlinear Static Procedure

Sections 6.5 through 6.13 present guidelines for specific concrete elements. These sections provide numerical recommendations for defining the nonlinear load-deformation relations.

C6.4.1.3 Flanged Construction

Tests and analysis show that both concrete and reinforcement within the monolithic flange of a beam or wall component act to resist tension and compression forces associated with flexure and axial load on the component (French and Moehle, 1991; Thomsen and Wallace, 1995). The effective flange width specified here is a crude measure of the effectiveness of the flange, to be used with the conventional Bernoulli assumption that plane sections remain plane. Action of the flange in tension—not included in current codes such as *ACI 318-95*—should not be overlooked. In general, the effect of the flange on the component is to increase bending and axial stiffness, increase bending and axial strength, and either increase or decrease flexural deformability depending on whether the flange is in compression or tension. The effects on the structure depend on details of the structure, but could include increased overall stiffness and strength, and modification of the yielding or failure mechanism. Consistent with conventional practice, a flange is considered ineffective in resisting shear out of its plane.

C6.4.2 Design Strengths and Deformabilities

C6.4.2.1 General

Acceptability criteria and strength specifications depend on whether a component has low, moderate, or high ductility demand, and whether the action is considered, according to Chapter 3, to be deformation-controlled or force-controlled.

Strength and deformability of reinforced concrete components are sensitive to details of geometry, reinforcement, materials, and load history including simulated gravity and earthquake loading. For example, flexural deformability is known to decrease with increasing nominal shear stress, all other factors being equal. Experiments must be designed to properly simulate important conditions. Expected variability in test results may sometimes be simulated analytically where suitable analytical models of the physical phenomena are available.

Reinforced concrete component resistance and deformation capacity tend to degrade with an increasing number of cycles and deformation levels. Degradation effects should be accounted for where numerous reversed loading cycles to large deformation levels are expected. These may be expected for structures with short periods and for structures subjected to long-duration ground motions. This effect should be considered primarily for deformability of deformation-controlled actions and for deformability and strength of force-controlled actions. Although strength degradation of deformation-controlled actions may occur, it usually is safer to disregard this degradation. The reason is that the forces in the deformation-controlled actions determine the design forces on the more brittle, force-controlled actions, and upper bound forces should be sought for design.

C6.4.2.2 Deformation-Controlled Actions

Deformation-controlled actions in reinforced concrete construction typically are limited to flexure and to shear in members with low aspect ratio. Flexure generally is the more ductile of the two, and resistance in flexure usually can be determined with greater accuracy. For this reason, deformation-controlled actions preferably will be limited to flexure.

As a flexurally-dominated component is flexed into the inelastic range, the longitudinal reinforcement in tension may be stressed to yield and beyond. The actual yield stress of reinforcing steel typically ranges from the nominal yield value up to about 1.3 times the nominal value, with average values about 1.15 times the nominal value. Tensile strength, which may be approached in components having high ductility demand, is typically 1.5 times the actual yield value. Therefore, the minimum recommended tensile stress of 1.25 times the nominal yield strength should be considered a low estimate suitable only for components with low and intermediate ductility demands.

C6.4.2.3 Force-Controlled Actions

In general, strengths Q_{CL} should be determined as realistically low estimates of component resistance over the range of deformations and coexisting actions to which the component is likely to be subjected. Where strengths are calculated, use low estimates of material strengths; however, assumed material strengths should be consistent with quantities assumed for deformation-controlled actions in cases where the same materials affect both strengths. For example, consider a reinforced concrete beam where flexural moment is the deformation-controlled action, and shear is the force-controlled action. In this case, beam flexural strength and beam shear strength are affected by concrete and reinforcement properties. It would be reasonable to calculate flexural strength assuming estimated concrete strength, and reinforcement stress equal to 1.25 times the nominal value. Shear strength would be calculated using the same assumed concrete strength and the same assumed nominal yield stress for the reinforcement, but without strain hardening. It would be unreasonable to assume a high compressive strength for flexure and a low compressive strength for shear, because the same concrete resists both actions.

C6.4.2.4 Component Ductility Demand Classification

Deformation ductility may be taken as displacement ductility, although it is conservative to use rotation or curvature ductility instead.

C6.4.3 Flexure and Axial Loads

Flexural strength calculation follows standard procedures, except that in contrast with some procedures, the developed longitudinal reinforcement in the effective flange width is to be included as tensile reinforcement. In existing construction, the longitudinal reinforcement may not be adequately developed at all sections. Where development length measured from a section is less than the length required to develop the yield stress, the stress used for strength calculation shall be reduced in proportion with the available length. Furthermore, the flexural deformability shall be based on the assumption of development failure, rather than flexural failure.

Flexural strength and deformation capacity of columns need to be calculated considering the axial forces likely to be coexisting with the design flexural demands. Except for conforming columns supporting discontinuous walls, where the column is in compression the flexural moment is a deformation-controlled action and the axial load is a force-controlled action. Where practicable, the column axial load should be determined by limit analysis or nonlinear analysis, as described in Chapter 3. The column flexural moment strength and corresponding acceptance criteria are then determined for this axial load. Where lateral loading in different directions results in different design axial loads, flexural strength and acceptability should be checked for both extremes and for critical cases in between. Special attention is required for corner columns, which may experience very high axial tension or compression for lateral loading along a diagonal of the building.

ACI 318-95 limits the maximum concrete compression strain for flexural calculations to 0.003. The same limit is permitted in the *Guidelines*. However, larger strains at the onset of concrete spalling are commonly achievable for components with significant strain gradients and components framing into adjacent blocks of concrete (for example, a column framing into a footing). The upper limit of 0.005 for unconfined sections is based on observed performance of components in laboratory tests. Larger calculated deformation capacities will result using this limit.

The compression strain limit of 0.005 for unconfined concrete is based on judgment gained through laboratory testing experience. When a component has a moment gradient, or when it frames into an adjacent component, the concrete is confined by adjacent concrete so that larger compression strains can be developed. The value 0.003 specified in the *ACI 318-95* Building Code is a lower-bound value that is intended to give a conservative estimate of strength for design of new construction. Larger values are used in some other codes for design of new structures.

The *Guidelines* permit the engineer to take advantage of the beneficial effects of concrete confinement provided by properly detailed transverse reinforcement (Sheikh, 1982). Appropriate details include close longitudinal spacing, cross-ties or intermediate hoops for wide sections, and anchorage into the confined core (or other appropriate means of anchoring the transverse steel). The analytical model for confined concrete should be consistent with the materials and details. The maximum usable compression strain of confined concrete may correspond to loss of component resistance due to either degradation of the confined concrete, fracture of transverse reinforcement, or buckling and subsequent fracture of longitudinal reinforcement. Buckling and subsequent fracture of longitudinal reinforcement appear to depend on both the maximum tensile strain and the maximum compressive strain experienced by the longitudinal reinforcement. At the time of this writing, accurate models for predicting this type of failure are not available. The recommended strain limits of 0.05 (tension) and 0.02 (compression) are based on observed performance of reinforced concrete components in laboratory tests, and are associated primarily with the phenomenon of reinforcement buckling and subsequent fracture.

Laboratory tests indicate that flexural deformability may be reduced as the coexisting shear force increases. At the time of this writing, analytical methods for considering effects of applied shear on flexural deformability are not well developed. The engineer should exercise caution when extrapolating results for low applied shear force to cases with high applied shear force.

C6.4.4 Shear and Torsion

Strength in shear and torsion has been observed to degrade with increasing number and magnitude of deformation cycles. Relations between shear strength and deformation demand have been proposed based on test results (Priestley et al., 1994; Aschheim and Moehle, 1992), but these are valid only within the loading regime used during the tests. The sequence and magnitude of inelastic deformations that will occur in a given building during an unknown earthquake cannot be predicted. Therefore, shear strength cannot be predicted accurately. The *Guidelines* therefore prescribe a simple procedure whereby for low ductility demands the strength is assumed to be equal to the strength for a nonyielding structure, and for other cases the strength is assumed to be equal to the strength expected for structures experiencing large ductility demand. For yielding components, it is permitted to calculate the shear strength outside flexural plastic hinges, assuming values for low ductility demand. For this purpose, the flexural plastic hinge length should be taken as equal to the section depth in the direction of applied shear.

To be effective in resisting shear, transverse reinforcement must be properly detailed and proportioned. The *Guidelines* specify minimum requirements.

The recommendation for shear friction strength is based on research results reported in Bass et al. (1989). The reduced friction coefficient for overhead work is because of poorer quality of the interface at this joint.

Additional information on shear strength and deformability is presented in the sections on concrete elements.

C6.4.5 Development and Splices of Reinforcement

Development of straight and hooked bars, and strength of lap splices, are a function of ductility demand and number of yielding cycles. General trends are similar to those described for shear in Section C6.4.4. For this reason, the specifications for development and lap splices are organized according to ductility demand.

For bars that are not fully developed according to the specifications of *ACI 318-95*, the bar stress capacity for strength calculations can be calculated as a linear function of the provided development or splice length. Where a bar has less than the development or splice length required for yield at a given section, and the calculated stress demand equals or exceeds the available capacity, development or splice failure should be assumed to govern. Splice failure should be modeled as a rapid loss in bar stress capacity.

The embedment length used in Equation 6-2 was derived from design equations in *ACI 318-95* that relate to pullout of bars having sufficient cover or transverse reinforcement, so that splitting of cover concrete cannot occur. The expression may be applied to bottom beam reinforcement embedded a short distance into a beam-column joint. For an embedment of six inches into a joint, which is common for frames designed for gravity loads only, Equation 6-2 typically produces values of f_s = 20 ksi or lower. Experimental research on beam-column connections (Moehle et al., 1994) indicates higher stress capacities may be available when flexural tension stresses in adjacent column bar reinforcement (which acts to clamp the embedded bar) are low. The available data support use of Figure C6-5 to estimate the stress capacity of the embedded bars. In Figure C6-5, the column longitudinal reinforcement stress is calculated based on column actions coexisting with the embedded bar tensile force.

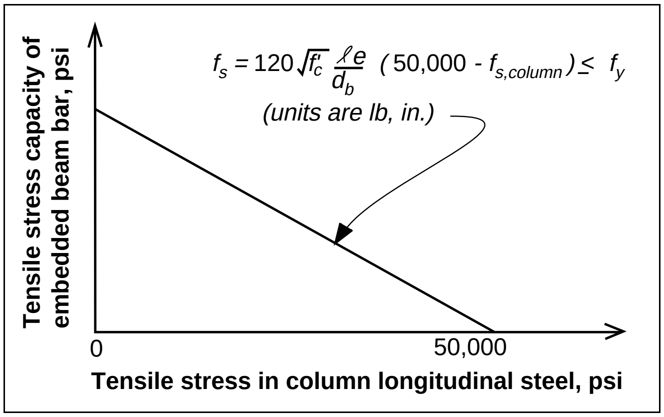


Figure C6-5 Relation Between Beam Embedded Bar Stress Capacity and Coexisting Tensile Stress in Adjacent Column Longitudinal Reinforcement

The specification for doweled bars is based on tests reported in Luke et al. (1985). Other suitable methods of anchoring new concrete to existing concrete are acceptable.

C6.4.6 Connections to Existing Concrete

Many different devices are used for attaching structural and nonstructural items to concrete. The design of anchorages has generally been based on engineering judgment, proprietary test data, manufacturers' data, and code requirements. Anchorage systems can be classified as either cast-in-place systems or post-installed systems.

C6.4.6.1 Cast-in-Place Systems

Anchors of this general classification come in a wide range of types and shapes, and utilize numerous attachment mechanisms. Typical examples are common bolts, hooked J or L bolts, threaded rod, reinforcing steel, threaded inserts, stud welded plates, and embedded structural shapes. The design of these anchoring components must consider the overall behavior of the connected components or elements and must consider the overall behavior of the anchorage. Anchorages are not only subject to shear and tensile forces, but also to bending and prying actions. The ductility and capacity of these connections should exceed the associated ductility of the connecting action as well as the magnitude of the action.

The location of the anchor with respect to potential cracking of the host concrete must be considered in the design. Edge distances, depth of embedment, spacing, and flexural cracking may reduce the capacity of the anchor by a factor of 0.5 or less. Consideration of the service environment is essential to reduce the potential of corrosion-induced failure.

ACI 355.1R-91 contains state-of-the-art information on anchorage to concrete. It is the first of a two-volume project being undertaken by ACI Committee 355; the referenced document emphasizes behavior, while the second volume is to be a design manual. Suggestions for design consideration and construction quality control are provided in the first volume. Designers are strongly encouraged to utilize this document in developing their anchorage designs. While this is not a code-like document, it provides a single point of reference for information needed for appropriate design.

C6.4.6.2 Post-Installed Systems

Anchors of this general classification include grouted anchors, chemical anchors, and expansion anchors. Excluded from consideration are powder-actuated fasteners, light plastic or lead inserts, hammer-driven concrete nails, and screen-driven systems. These are excluded because there is little test data to recommend their use.

The commentary for this section includes the material in Section C6.4.6.1. An additional item to be considered is that anchors of this type generally have little ductility associated with their behavior. They therefore should be designed for the total unreduced demand associated with the connected components.

Test data and design values for various proprietary post-installed systems are available from various sources. Because there commonly is a relatively wide scatter in ultimate strengths, common practice is to define working loads as one-quarter of the average of the ultimate test values. Where working load data are defined in this manner, it may be appropriate to use a design strength equal to twice the tabulated working load. Alternatively, where ultimate values are tabulated, it may be appropriate to use a design strength equal to half the tabulated average ultimate value. The implicit objective of these suggestions for design strengths is to define the design strength as the lower-bound strength of Chapter 3. Accordingly, where statistical data are available the design strength may be taken at the lower five-percentile value.

C6.4.6.3 Quality Control

Connections between seismic resisting components must be subjected to a high level of installation inspection and testing. Many different installation factors can greatly reduce the expected capacities of all connection systems. ACI Report 355.1R-91 provides guidance with respect to this issue. Special care must be taken by the design professional specifying the inspection and testing of anchorage and connection systems.

The design of post-installed systems is susceptible to being altered in the field, due to existing reinforcing steel. Magnetic and radiographic procedures are available to help in locating conflicting reinforcing steel during the design stage, but all conditions and variations are difficult to predetermine. Contingency plans should be made as to how to deal with conflicts in anchor placement. Rebar should rarely be cut and then only under the direction of the engineer of record.

C6.5 Concrete Moment Frames

C6.5.1 Types of Concrete Moment Frames

Properly-proportioned and detailed reinforced concrete frames can provide an efficient system for resisting gravity and lateral loads, while providing maximum flexibility for use of interior spaces. To function properly in resisting earthquake effects, the framing system should provide at least the following:

- **Adequate stiffness.** Stiffness is important in controlling lateral displacements during earthquake response to within acceptable limits. While the *Guidelines* do not impose general limits on lateral drift ratios for all materials of construction, some guidance on target drift levels is provided in Table 2-4. The target drift levels suggested in the table are derived from experience with successful performance of buildings in past earthquakes; significant deviations above these limits should only be accepted after careful consideration. Lateral drift also needs to be limited to avoid pounding with adjacent structures, per Section C6.2. As noted in Section C6.2, pounding of adjacent buildings, especially when floor levels for the pounding buildings do not align, may lead to severe damage to impacted columns, and may cause collapse. Excessive lateral drift may also contribute to second-order P-Δ effects associated with gravity loads acting through lateral displacements. Some additional restrictions on lateral drifts are imposed in Chapter 11, because of the potential for damage to nonstructural components and contents.

- **Proper relative proportions of framing components.** To function properly, it is desirable that inelastic action, if it occurs, be distributed throughout the structure rather than being concentrated in a few components. In reinforced concrete frames, this usually is achieved by providing a stiff, nonyielding spine throughout the building height. This spine can be either a stiff reinforced concrete wall that is continuous through the building height, or the columns themselves if they are sufficiently strong. If the columns are made stronger than the horizontal framing members, yielding will tend to occur primarily in the beams, ideally resulting in a beam sway mechanism in which horizontal framing components yield throughout the building height (Figure C6-6b). On the other hand, if the columns are weaker than the horizontal framing components, yielding will tend to concentrate in a single story, possibly leading to a column sway mechanism (Figure C6-6a). This latter failure mechanism is one of the prominent causes of collapse in reinforced concrete building construction. Attention also must be paid to strength of beam-column connections. In general, it is desirable that connections be made stronger than the adjacent framing components. Beam-column joint failures, especially for exterior and corner

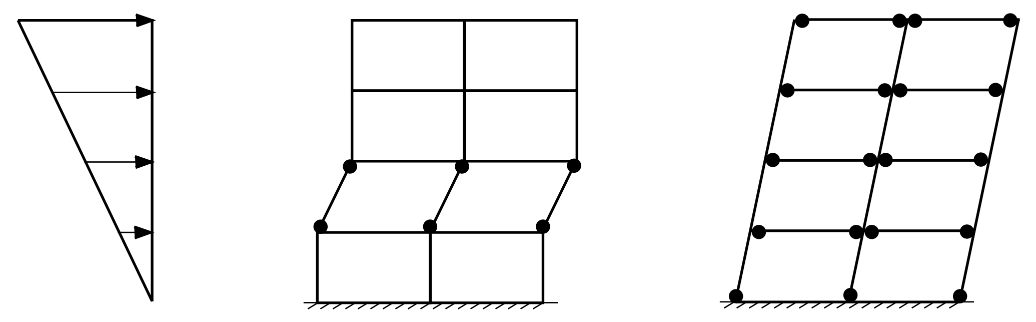


Figure C6-6 Flexural Failure Mechanisms of Reinforced Concrete Frames

connections, have contributed to many building collapses in past earthquakes.

- **Adequate detailing.** Framing components need to be detailed with reinforcement that provides them with adequate toughness. In both columns and horizontal framing components, the longitudinal reinforcement needs to be reasonably continuous and well-anchored, so that flexural tension stresses can be resisted under the full range of flexural moments that will be experienced during a design-level event. Lap splices preferably will be located away from locations of inelastic flexural action, or will be confined by closely spaced, well-detailed transverse reinforcement. Transverse reinforcement spacing and detailing should be adequate to confine wherever compression strains are large (that is, where axial loads are high or where flexural plastic hinges require large rotation capacity). Transverse reinforcement also should be proportioned and detailed to prevent shear failures in columns and beams. Where joints are heavily stressed, joint transverse reinforcement also is an essential element of a tough framing system. The literature abounds with documentation of building collapses associated with failures of inadequately detailed columns and joints. Beam failures do not appear to have been a major cause of building collapse in past earthquakes, but adequate attention to their details is nonetheless important in design.

C6.5.1.1 Reinforced Concrete Beam-Column Moment Frames

Where new frames are added as part of a seismic rehabilitation, it is preferable that they satisfy the requirements for Special Moment Frames, Intermediate Moment Frames, or Ordinary Moment Frames, whichever is appropriate according to definitions and requirements of the *NEHRP Recommended Provisions* (BSSC, 1995). However, because of constraints imposed by existing conditions, it may not be possible to satisfy all requirements for these predefined framing types. Because design requirements have evolved continually, it is unlikely that any existing frame will fully comply with the requirements of modern codes. For example, many older existing frames will satisfy many—but not all—of the provisions required for new ordinary moment frames. For these reasons, the terms “Special Moment Frame,” “Intermediate Moment Frame,” or “Ordinary Moment Frame” are not used broadly in the *Guidelines*.

Some existing bearing wall buildings may rely on wall resistance for loading in the plane of the wall, and on slab-wall framing for loading out of the plane of the wall (the wall acts as a wide column in this loading direction). The slab-wall frame, loaded out of the plane of the wall, may be classified as a beam-column moment frame.

C6.5.1.2 Post-Tensioned Concrete Beam-Column Moment Frames

This classification excludes precast construction that is pretensioned or post-tensioned, which is covered by Section 6.6 of the *Guidelines*.

C6.5.1.3 Slab-Column Moment Frames

In certain parts of the United States, it is common practice to design slab-column frames for gravity loads alone and to assign lateral load resistance to other elements, such as beam-column moment frames and shear walls. Slab-column frames designed according to this practice are included within the scope of Section 6.5, as it may be possible to derive some benefit in lateral load resistance from these frames, and because these frames should be analyzed to ensure that they continue to support gravity loads under the design lateral deformations.

C6.5.2 Reinforced Concrete Beam-Column Moment Frames

C6.5.2.1 General Considerations

The main structural components of beam-column frames are beams, columns, and beam-column connections. The beam may be cast monolithically with a reinforced concrete slab, in which case the slab should be considered to act as a flange of the beam.

Experience in earthquakes demonstrates that frames, being relatively flexible, may be affected negatively by interaction with stiff nonstructural components and elements. The analytical model should represent this interaction.

Provisions for design of new buildings (e.g., ACI 318) are written so that inelastic action ideally is restricted to flexure at predetermined locations. Inelastic action in an existing building may be by flexure at sections other than the component ends, by shear or bond failure, or by some combination of these. The analytical model should be established recognizing these possibilities. Usually it is preferable to establish the likely inelastic response of a component using free-body diagrams of the isolated component rather than relying on the complete structure analysis model for this purpose. This approach is illustrated in Section C6.4.1.1.

The recommendations for eccentric connections are based largely on practical considerations and engineering judgment. Some tests have investigated this condition (Joh et al., 1991; Raffaelle and Wight, 1995).

Some tests on beam-column joints having beams wider than columns have been reported (Gentry and Wight, 1994). These indicate that wide beams can be effectively connected to columns, given certain details.

The restrictions on types of inelastic deformation are based on the observation that lateral load resistance cannot be sustained under repeated loadings for frame members whose strength is controlled by shear, torsion, or bond. Some inelastic response in shear, torsion, or bond may be acceptable in secondary components, which by definition are required only for gravity load resistance.

C6.5.2.2 Stiffness for Analysis

A. Linear Static and Dynamic Procedures

No commentary is provided for this section.

B. Nonlinear Static Procedure

Available inelastic models for beams include concentrated plastic hinge models, parallel component models, and fiber models (Spacone et al., 1992). With plastic hinge models, inelastic behavior is restricted to those locations where the analyst has placed nodes in the analytical model, typically at beam ends adjacent to the columns. If inelastic response is possible at other locations along the beam span, it is necessary to subdivide the beam into shorter segments having potential plastic hinges located at the end of each segment. Usually a beam can be evaluated separately before assembling the complete structure model to determine if internal plastic hinges are likely (see Section C6.4.1.1).

Reinforced concrete columns can be modeled using the same models identified for beams, except that where there are significant axial force variations under the action of earthquake loading, the model should also represent the effects of that variation on stiffness and strength properties. This is possible using interaction surfaces for plastic hinge models. Fiber models usually can represent this effect directly.

C. Nonlinear Dynamic Procedure

Hysteretic relations used for the NDP should resemble the response obtained for reinforced concrete components. It is preferable that nondegrading bilinear relations not be used. Simple stiffness degrading component models such as the Takeda and Modified Clough relations (Saiidi, 1982) are preferred. Figure C6-7 is a sample of a load-deformation relation produced by the Takeda model. The model features reduced stiffness beyond yield and stiffness degradation with increasing displacement amplitude. For existing construction with inadequate details, there may be strength degradation in addition to stiffness degradation. Some hysteretic models including stiffness and strength degradation have been reported (Kunnath et al., 1992). The rate of strength degradation for these models needs to be calibrated with experimental data.

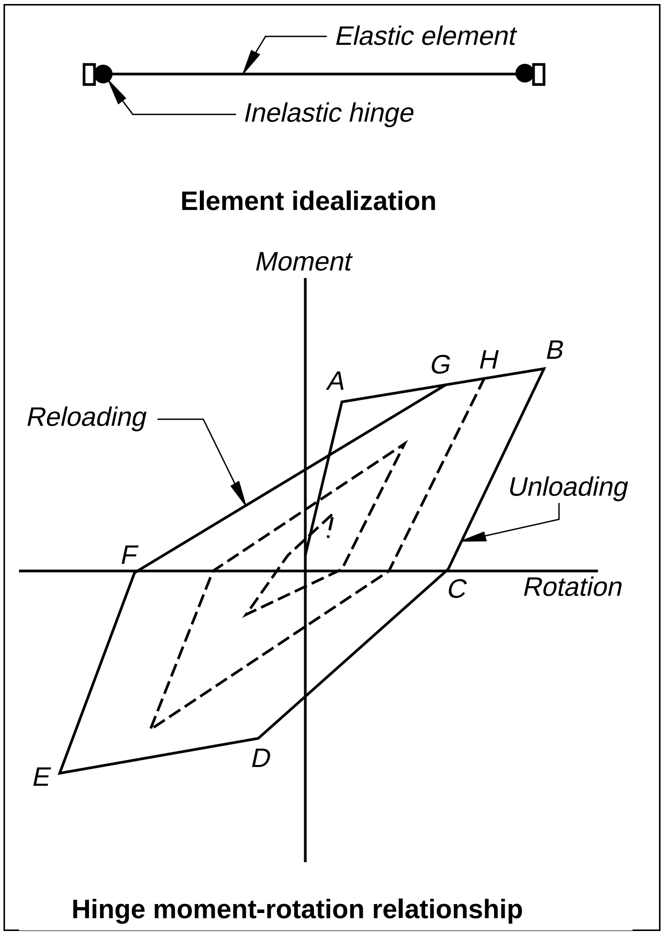


Figure C6-7 Takeda Hysteresis Model

Figure C6-8 presents some typical load-deformation relations measured during laboratory tests of reinforced concrete components. These illustrate a range of performances that might be anticipated. The relations shown should not be construed as being representative of components in existing construction, but should be used only as a guide in selecting general characteristics of hysteretic models.

C6.5.2.3 Design Strengths

As described in Section 6.4.2, component strengths are calculated based on procedures from *ACI 318-95*, with some modification to reflect differences in details and proportions, as well as to reflect the different purposes of the *ACI 318-95* document and the *Guidelines*.

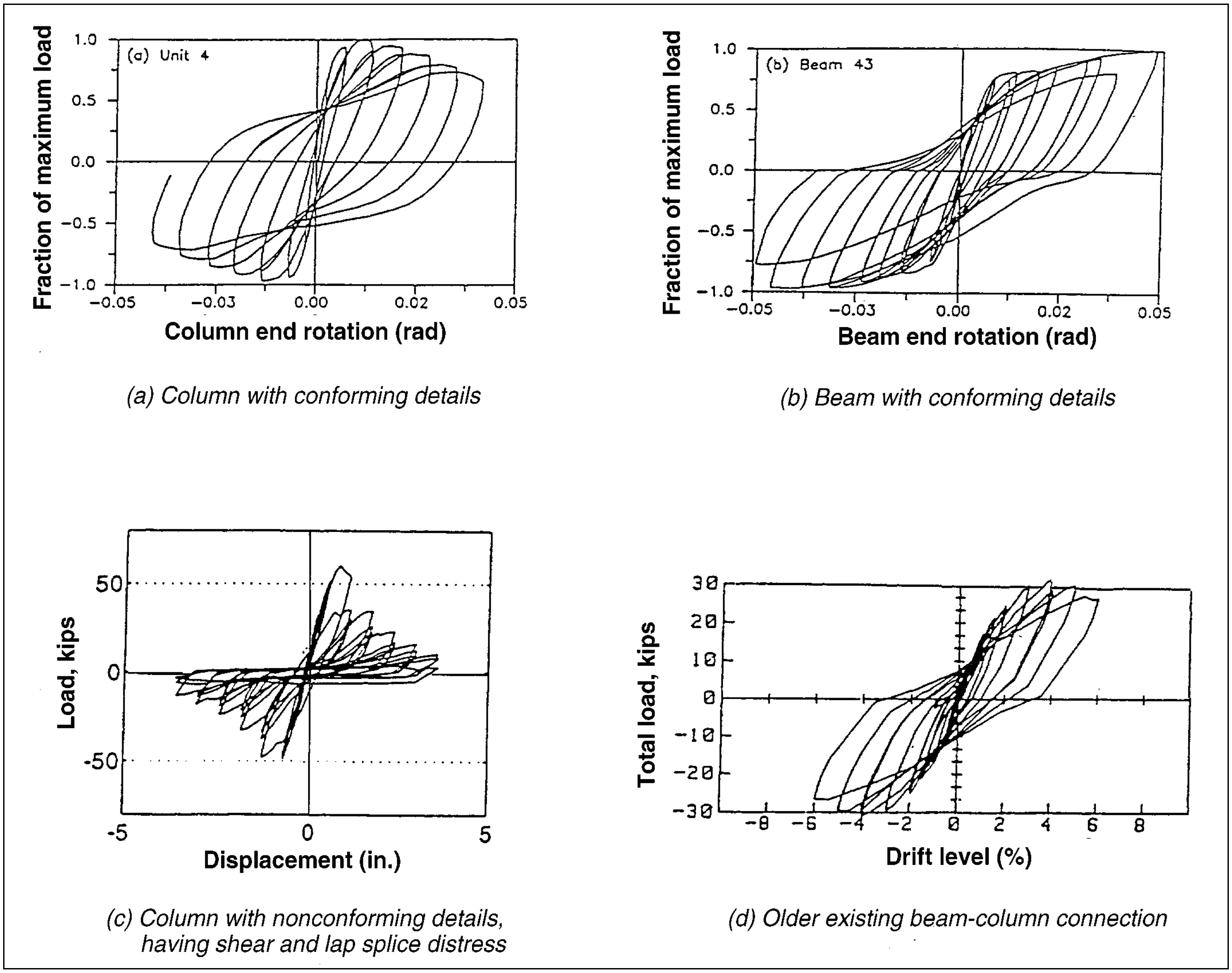
The engineer is reminded that inelastic response and failure may occur in any of a number of different modes, and may occur at any section along the length of the component, including its connections.

Experiments on columns subjected to axial load and reversed cyclic lateral displacements indicate that *ACI 318-95* design strength equations may be excessively conservative for older existing columns, especially those with low ductility demands (Priestley et al., 1994; Aschheim and Moehle, 1992). The recommended column shear strength equation is based on a review of the available test data. The available strength in older columns is strongly related to ductility demand; therefore, conservative procedures should be used to determine whether ductility demands will reach critical levels. The distinction between low ductility demand and moderate or high ductility demand is discussed in Section 6.4.2.4. The restriction on axial loads calculated using the linear procedures of Chapter 3 is based on the understanding that the axial load calculated using linear procedures may overestimate the axial load in a yielding building. The restriction will produce conservative effects. The axial load preferably should be calculated using limit analysis procedures as described in Section 3.4.2.1B. Simple procedures involving summation of the beam plastic shears are appropriate for this purpose.

Shear failure in columns is a common source of damage and collapse in older buildings. Engineering judgment should be applied—in addition to the specifications of the *Guidelines*—to determine the proper course of action for buildings with columns having widely-spaced ties and moderately high shear stresses.

The specification for beam-column joint shear strength is developed from various sources. Kitayama et al. (1991) and Otani (1991) present data indicating that joint shear strength is relatively insensitive to the amount of joint transverse reinforcement, provided there is a minimum amount (a transverse steel ratio equal to about 0.003). Beres et al., (1992a) report on shear strengths of joints without transverse reinforcement. Although some researchers report that increased column axial load results in increased shear strength, the data do not show a significant trend.

Design actions (axial loads and joint shears) on beam-column joints preferably should be calculated from consideration of the probable resistances at the locations for nonlinear action. Procedures for



(a) Column with conforming details

(b) Beam with conforming details

(c) Column with nonconforming details, having shear and lap splice distress

(d) Older existing beam-column connection

Figure C6-8 Sample Load-Deformation Relations for Reinforced Concrete Beams, Columns, and Beam-Column Connections

estimating joint shear are the same as those specified in *ACI 318-95*.

C6.5.2.4 Acceptance Criteria

A. Linear Static and Dynamic Procedures

The basic acceptance criteria of Chapter 3 require that all actions be classified as either displacement-controlled or force-controlled actions. For beam-column moment frames, it is preferred that deformation-controlled actions be limited to flexure in beams, although some flexural yielding in columns (at least at the foundation level) is usually inevitable. This preference lies in the observation that beams yielding in flexure usually have moderate to high ductility capacities. Column flexural yielding is usually less ductile because of the detrimental effects of axial loads on deformability, and because excessive yielding in columns may lead to story sway mechanisms (see Section C6.5.1). Low-ductility capacity response modes—such as shear, torsion, or reinforcement development or splicing of beams or columns, and shear in beam-column joints—are to be avoided in primary components designed using the linear procedures of Chapter 3. Yielding in some low-ductility capacity response modes is permitted in secondary components where gravity loads are likely to be sustained through moderate levels of ductility demand.

Tables 6-10 through 6-12 present allowable values for these secondary component cases.

Ideally, where linear procedures are used for design, the actions obtained directly from the linear analysis will be used only for determining design values associated with yielding actions in the structure. The design actions in the rest of the structure should be determined using limit analysis procedures considering the gravity forces plus the yielding actions acting on a free body diagram of the component or element. The *Guidelines* specify actions that should be designed on this basis.

Reinforced concrete components whose design forces are less than force capacities can be assumed to satisfy all the performance criteria of the *Guidelines*. However, it is still necessary to check performance of all other components and the structure as a whole.

Beam-column frames with widely-spaced column transverse reinforcement may be susceptible to story collapse due to column failure. Column shear failure can initiate the collapse if shear capacity is less than shear strength demand. Flexural failure can initiate the collapse if inelastic column flexural demands lead to strength degradation. Frames having columns with flexural strengths less than the adjoining beam flexural strengths are particularly vulnerable to this latter type of failure. To minimize the likelihood of this type of failure in new construction, codes for new building construction require that column end regions contain copious amounts of transverse reinforcement, and that the sum of strengths of columns exceed the sum of strengths of beams at each joint. With a similar objective, the *Guidelines* specify that DCR values for beams and columns be checked (which is similar to checking relative strengths) and that DCR values be compared with DCR capacities (a conservative measure of $m/2$ is specified). The check is carried out as an average for all components at the floor level being checked, rather than at each connection as specified in *ACI 318-95*. Where an element fails the check, either: (1) the check is repeated for all elements of the system, since story collapse is likely to involve more than one frame; (2) the structure is reanalyzed by one of the nonlinear approaches, which is likely to provide an improved measure of the actual demands; or (3) the structure is rehabilitated to remove the deficiency.

The m values in Tables 6-6, 6-7, and 6-8 were developed from the experience and judgment of the project team, guided by available test data (Aycardi et al., 1994; Beres et al., 1992; Lynn et al., 1994; Pessiki et al., 1990; Qi and Moehle, 1991).

B. Nonlinear Static and Dynamic Procedures

Inelastic response preferably will be limited to flexure in beams and columns. For components whose strength is limited by shear, torsion, and reinforcement development and splicing, the deformability usually is less than for flexure, and stability under repeated deformation cycles is often questionable. Where inelastic action other than flexure is permitted, it is preferable that it be limited to a few components whose contribution to total lateral load resistance is a minority.

Inelastic action is not desirable for actions other than those listed in Tables 6-6, 6-7, and 6-8. Where inelastic response is acceptable, calculated deformations should not exceed the deformation capacities listed in Tables 6-6, 6-7, and 6-8.

C6.5.2.5 Rehabilitation Measures

The rehabilitation strategies and techniques listed in the *Guidelines* are intended to provide guidance on procedures that have been successfully used for seismic rehabilitation of reinforced concrete beam-column moment frames. The list is not intended to exclude alternate procedures that are demonstrated to be effective in satisfying the Rehabilitation Objective. A summary of past research on rehabilitation techniques for reinforced concrete frames is provided by Moehle et al. (1994); Sugano (1981); and Rodriguez and Park (1991).

Commentary on the noted rehabilitation schemes is provided below.

- **Jacketing existing beams, columns, or joints with new steel or reinforced concrete overlays.** Jacketing may serve to increase flexural strength and ductility, and shear strength; to improve longitudinal reinforcement development or splicing; and to combine these effects. Although jacketing can be a technically effective procedure, when several components must be jacketed, it may not be cost-effective, and it can also be very disruptive to building occupants.

  Where jackets are used to increase flexural strength, and in some other cases requiring composite action, appropriate measures should be implemented to provide shear transfer between new and existing materials. These measures may include:

- For concrete jackets, roughening the surface of the existing concrete prior to concrete placement, and using dowels to improve shear transfer strength when the jacket does not surround the component
- For steel jackets, using epoxy to effectively bond the steel to the concrete, and nonshrink grout or dry pack plus bolts or other anchorage devices

Where the objective is to increase component flexural strength, the technique must provide continuity across beam-column connections so that the enhanced strength can be transferred to adjacent framing components (Alcocer and Jirsa, 1993; Corazao and Durrani, 1989; Rodriguez and Park, 1992; Krause and Wight, 1990; Stoppenhagen and Jirsa, 1987). For columns, approaches include the following:

- New longitudinal reinforcement can be passed through the floor system and encased in a reinforced concrete jacket.
- Steel sections flanking the existing column can be connected to it to ensure composite action, and pass through the floor system to provide continuity. Similar approaches may be used for beams, including the addition of straps or continuous reinforcement across joints where beam bottom reinforcement is discontinuous.

Where the objective is to increase flexural ductility, either reinforced concrete or steel jackets can be added to deficient sections (Aboutaha et al., 1994). If the jacket completely surrounds the component or, in the case of beams, the jacket surrounds three faces and is anchored into the slab, only a nominal connection is required between existing and new materials. Concrete jackets should be reinforced with transverse reinforcement and nominal longitudinal reinforcement. Steel jackets may comprise bands or full-height jackets made of steel plates or shells; anchorage may be necessary along the side face of flat steel plates to improve confining action, and stiffeners may be required for thin plates. The space between steel jackets and existing concrete should be filled with nonshrink grout. If the purpose of the jacket is to increase the flexural ductility but not increase the flexural strength, the longitudinal reinforcement in concrete jackets and steel in steel jackets should be discontinued a short distance (about 50 mm) from the connection with adjacent components. Concrete jackets placed to improve ductility may also enhance flexural strength, which may shift the ductility demands to adjacent sections, and this aspect should be checked and appropriate actions taken. In general, a jacket should extend from critical sections a distance equal to at least 1.5 times the cross-sectional dimension measured in the direction of the lateral load.

Where the objective is to increase shear strength, steel, concrete, or other types of jackets can be added to deficient sections (Bett et al., 1988; Katsumata et al., 1988; Aboutaha et al., 1993). The general approach to designing the jacket and its connection with the existing concrete is similar to that described in the preceding paragraph. When proper connections between old and new materials are achieved, it is usually appropriate to calculate the nominal shear strength as if the section were composite.

Where the objective is to improve performance of inadequate reinforcement development or splicing, either reinforced concrete or steel jackets may be used (Aboutaha et al., 1994). The jackets should be designed to restrain splitting action associated with development or splice failure. Concrete jackets require transverse reinforcement and may require cross ties; steel jackets may require bolts anchored into the concrete core.

Where the objective is to improve continuity of beam bottom reinforcement, supplementary reinforcement may be added to improve continuity (Beres et al., 1992b).

- **Post-tensioning existing beams, columns, or joints using external post-tensioned reinforcement.** Post-tensioning may serve to increase flexural strength and shear strength of beams and columns. It may also reduce deficiencies in reinforcement development and splicing if tension stress levels are reduced. Joint shear strength may also be enhanced by joint post-tensioning.

  Usually it is preferable to not bond the post-tensioned reinforcement in regions where inelastic response is anticipated. Bonded reinforcement is more likely to undergo inelastic strain that may relieve the post-tensioning stress. Anchorage zones should also be placed away from inelastic regions because of the potential for anchorage damage in these regions.

- **Modifying of the element by selective material removal from the existing element.** Partial or full-height infills in existing beam-column frames may have inadequate separation between the infill and the concrete frame. In some cases, it is desirable to use the infill as a structural component (see Section 6.7). In other cases, it is desirable to separate the infill from the concrete frame so that lateral resistance is provided by beam-column framing. Either the infill can be entirely removed, or the joint between the infill and the frame can be cleaned and filled with flexible jointing material. In the latter case, the joint dimension should be at least equal to the interstory drift calculated using the Analysis Procedures of Chapter 3.

  Other architectural components that may affect the structural framing include stairs and nonstructural exterior curtain walls. In some cases, gaps can be increased or rigid connections can be replaced with flexible connections to reduce the interaction with the structural framing.

  Beams and columns can also be selectively weakened to improve structural performance. For example, beam longitudinal reinforcement or section depth can be reduced to weaken the beam, thereby promoting development of a strong column-weak beam framing action. Beam and column longitudinal reinforcement can also be severed to decrease shear demands associated with flexural hinging of these components. Weakening of existing structural components is often considered unacceptable, even if this action promotes improved overall behavior of the building. When considering weakening of a structural component, the impact on safety and serviceability under design load combinations—including gravity load, and gravity load plus design lateral loads—should be evaluated.

- **Improving deficient existing reinforcement details.** This approach does not include jacketing, which is covered elsewhere. As with jacketing, this approach may not be cost-effective, and may be overly intrusive.

  This approach may be effective where reinforcement lap splices or anchorages are inadequate. The approach in this case is to remove cover concrete, lap weld existing reinforcement together or weld auxiliary reinforcement between adjacent inadequately developed bars, and replace concrete cover.

  This approach has also been used to add transverse reinforcement to confine inadequately confined lap splices, but tests have shown that this technique may be ineffective. Transverse reinforcement can be added effectively to improve shear strength.

- **Changing the building system to reduce the demands on the existing element.** This approach involves reducing the displacement demands on the existing element by adding new vertical elements (such as moment frames, braced frames, or walls), by adding seismic isolation or supplemental damping, or by otherwise modifying the building. Approaches to changing the building system to reduce seismic demands are discussed in Chapter 2.

- **Changing the frame element to a shear wall, infilled frame, or braced frame element by addition of new material.** This approach usually involves filling openings with reinforced concrete (Altin et al., 1992) or adding steel bracing components to convert the existing moment frame to a shear wall or braced frame (Bush et al., 1991; Goel and Lee, 1990). Where wall openings are filled with concrete, two approaches have been considered. In the first, the entire opening is filled, converting the panel to a structural wall. In the second, a portion of the opening on each side of the existing column is filled to transform the existing column to a wall pier (the added portions of concrete are commonly referred to as wing walls—see Bush et al., 1990). Decisions about how to modify frames, and which ones to modify, depend partly on technical issues and partly on nonstructural considerations.

Where openings in frames are filled with reinforced concrete, at least the following aspects should be considered:

– The wall panel should be designed according to requirements for new wall construction. Wall panel reinforcement should be doweled into existing beam and column sections, to transfer tensile forces from wall reinforcement and to provide shear transfer between new and old concrete.

– Wall boundary reinforcement should be provided where necessary (Jirsa and Kreger, 1989). Where the infill fills the entire opening and the wall panel is adequately connected to the columns, the columns may act as boundary elements. The adequacy of column transverse reinforcement and longitudinal reinforcement (including lap splices) to transfer required forces and sustain required deformations should be checked. Columns may be jacketed to improve their adequacy. Additional wall vertical reinforcement (distributed or concentrated near the boundaries) can be provided. Usually the additional reinforcement can pass through the floor system adjacent to the beam webs.

– If some of the openings in the frame are not filled, the effect of the new wall panel on the existing unfilled portions should be checked.

– The floor diaphragm, struts, and collectors are to be checked to ensure that there is an adequate system to transfer lateral forces to the new wall element. They may be enhanced if necessary.

– The foundation is to be checked to be certain it is capable of resisting both the extra weight of the new material and the increased overturning and shearing actions beneath the rehabilitated element.

- Where steel bracing is provided in existing concrete moment frames, at least the following aspects should be considered:

  – The bracing components should be designed according to accepted practices for steel bracing.

  – Steel braces should be connected to the existing concrete frame to transfer the design forces. The attachment details should be designed to minimize the impact on the existing concrete materials.

  – Adequacy of the existing concrete frame components (beams and columns) to transfer actions developed in the rehabilitated element should be evaluated. Adequacy of column transverse reinforcement and longitudinal reinforcement (including lap splices) to transfer required forces and sustain required deformations should be checked. Columns may be jacketed to improve adequacy. Steel strapping to supplement capacity is permitted.

  – The effects of the new bracing system on the existing frame, including portions not provided with braces, should be checked.

  – Collectors and floor diaphragms are to be checked to ensure that they are capable of transferring lateral forces to the new braced frame element. They may be enhanced if necessary.

  – The foundation is to be checked to be certain it is capable of resisting both the extra weight of the new material and the increased overturning and shearing actions of the rehabilitated element.

Post-tensioning steel can also be considered for lateral bracing of deficient buildings (Miranda and Bertero, 1991; Pinchiera and Jirsa, 1992).

C6.5.3 Post-Tensioned Concrete Beam-Column Moment Frames

C6.5.3.1 General Considerations

The limiting conditions presented in Section 6.5.3.1 are the same as those described in the *NEHRP Recommended Provisions* (BSSC, 1995) for new buildings with prestressed and nonprestressed reinforcement. As documented by Ishizuka and Hawkins (1987), if these conditions are met in new buildings the seismic design provisions for nonprestressed moment frames apply. The recommendation of the *Guidelines* is to extend this finding to existing construction. Satisfactory seismic performance can be obtained in frames using prestressing amounts greater than those listed in

Section 6.5.3.1, but reductions in allowable *m* values or inelastic deformation values may be required. Relevant discussion may be found in Park and Thompson (1977) and Thompson and Park (1980).

BSSC (1995) recommends for new buildings that anchorages for tendons be capable of withstanding, without failure, a minimum of 50 cycles of loading ranging between 40 and 85% of the minimum specified tensile strength of the tendon. It also recommends that tendons extend through exterior joints and be anchored at the exterior face or beyond. These recommendations apply also to the *Guidelines*.

C6.5.3.2 Stiffness for Analysis

A. Linear Static and Dynamic Procedures

No commentary is provided for this section.

B. Nonlinear Static Procedure

It is assumed that a prestressed concrete beam behaves in a manner equivalent to a nonprestressed beam when conditions (1), (2), and (3) of Section 6.5.3.1 are satisfied. When these conditions are not satisfied, behavior parameters are to be derived from experiments or rational analysis.

C. Nonlinear Dynamic Procedure

Prestressing may result in component hysteresis that is markedly different from that for nonprestressed reinforced concrete components. Figure C6-9 presents some examples. The analytical model should represent the relevant characteristics of the load-deformation response.

C6.5.3.3 Design Strengths

A yielding prestressed concrete flexural member will develop strength associated with force levels developed in prestressed and nonprestressed reinforcement. Yielding of prestressed reinforcement may result in loss of prestress upon load reversal. The effects of this loss on the strength of force-controlled actions should be considered.

C6.5.3.4 Acceptance Criteria

No commentary is provided for this section.

C6.5.3.5 Rehabilitation Measures

The general rehabilitation procedures of Section 6.5.2.5 apply to prestressed concrete frames. Where seismic rehabilitation involves modification of the existing prestressed frame, including attachment of new materials, care should be taken to avoid damage to existing prestressing tendons and anchorages.

C6.5.4 Slab-Column Moment Frames

C6.5.4.1 General Considerations

The main structural components of slab-column frames are slabs, columns, slab-column joints, and the slab-column connection. In most cases, slab-column joints are not critical; therefore, no further discussion on slab-column joints is included in Section 6.5.4. Relevant material on beam-column joints should be referred to for special cases where slab-column joints may have high shear stresses. The slab-column connection commonly is a critical component in the system. It comprises the region of slab immediately adjacent to the column. Shear failure of the slab associated with shear and moment transfer can result in progressive collapse in cases where slab bottom reinforcement (or post-tensional strand) is not continuous through the column (see the report *ACI 352* [ACI, 1988] for further information). Beams are common around the perimeter of buildings that otherwise have predominantly slab-column framing. This case is covered in Section 6.5.4.

As with beam-column frames, experience indicates that slab-column frames may be affected negatively by interaction with nonstructural components and elements. The analytical model should represent this interaction.

Provisions for design of new buildings (e.g., *ACI 318-95*) are written so that inelastic action is restricted, ideally, to flexure at predetermined locations. Inelastic action in an existing building may be by flexure at sections other than the component ends, by shear or bond failure, or by some combination of these. The analytical model should be established recognizing these possibilities. Usually it is preferable to establish the likely inelastic response of a component using free-body diagrams of the isolated component rather than relying on the complete structure analysis model for this purpose. This approach is illustrated in Section C6.4.1.1.

Analytical models for slab-column frames usually are one of three types, illustrated in Figure C6-10. The effective beam width model (Figure C6-10b) represents the slab as a flexural member having stiffness reduced to represent the indirect framing between slab and

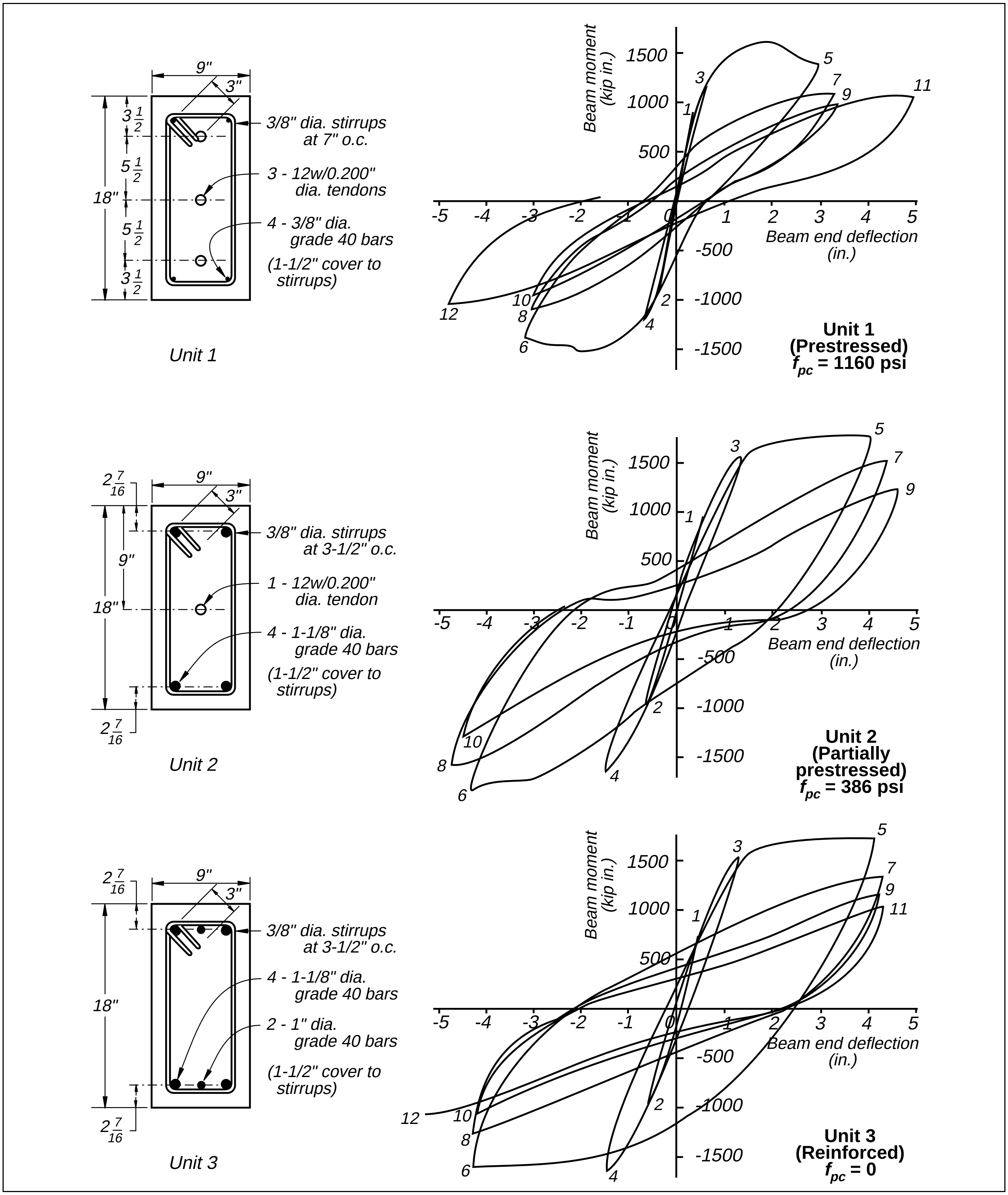


Figure C6-9 Sample Load-Deformation Relations for Prestressed, Partially-Prestressed, and Reinforced Beams column, as well as slab cracking. The equivalent frame model (Figure C6-10c) represents the slab by a flexural member that connects to the column through a transverse torsional member. Direct finite element models (Figure C6-10d) represent the flexural, shear, and torsional response of the slab directly. For each of the three models, the stiffness should be adjusted from theoretical values based on the gross cross section because of the significant effects of slab cracking on response (Vanderbilt and Corley, 1983). The effective beam width model, while simple to use, has a drawback in that there is no component to monitor directly the shear and moment transfer between slab and column, and this is an important aspect in checking performance. The finite element model has certain advantages, but has a relatively high computational cost. In most cases, it may be preferable to use an equivalent frame model because it provides a component to directly monitor shear and moment transfer.

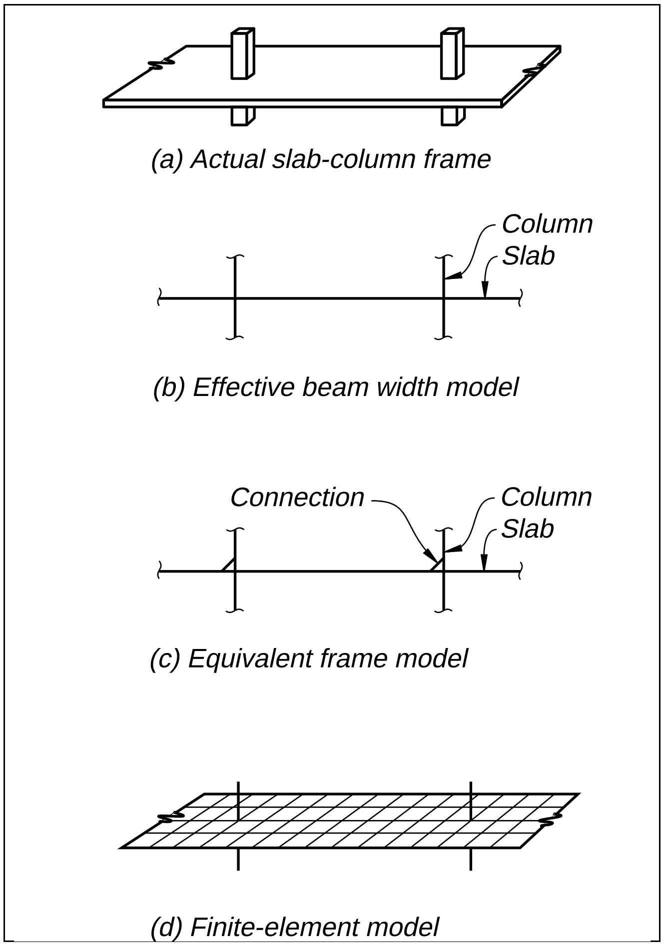


Figure C6-10 Models for Slab-Column Framing

The restriction on types of inelastic deformation are based on the observation that lateral load resistance cannot be sustained under repeated loadings for frame members whose strength is controlled by shear, torsion, or bond. Some inelastic response in shear, torsion, or bond may be acceptable in secondary components, which by definition are required only for gravity load resistance.

C6.5.4.2 Stiffness for Analysis

A. Linear Static and Dynamic Procedures

Any of the three models depicted in Figure C6-10, and other validated models, may be used to represent the slab-column frame. Whatever the model, it is essential to take into account the reduction in framing stiffness associated with cracking of the slab near the column. This cracking can reduce the effective linear elastic stiffness to as little as one-third the uncracked value (Vanderbilt and Corley, 1983; Pan and Moehle, 1992; Hwang and Moehle, 1993). Further discussion follows.

Various approaches to representing effects of cracking on stiffness of reinforced concrete slabs have been proposed and verified. Vanderbilt and Corley (1983) recommend modeling the slab-column frame using an equivalent frame model (Figure C6-10c) in which the slab flexural stiffness is modeled as one-third of the gross-section value. Hwang and Moehle (1993) recommend an effective beam width model (Figure C6-10b) having an effective width for interior framing lines equal to $\beta(5c_1 + 0.25l_1)$, where β represents cracking effects and ranges typically from one-third to one-half, c_1 = column dimension in the direction of framing, and l_1 = center-to-center span in the direction of framing. For exterior frame lines, half this width should be used. Note that this effective width applies only where the analysis model represents the slab-column joints as having zero horizontal dimension. Alternate approaches may be used where verified by tests.

For prestressed slabs, less cracking is likely, so it is acceptable to model the framing using the equivalent frame model without the factor one-third, or the effective width model with $\beta = 1.0$.

B. Nonlinear Static Procedure

It is essential that the nonlinear analysis model represent the behavior of the slab-column connection in addition to the slab and column components. Nonlinear response of slab-column connections is a complex

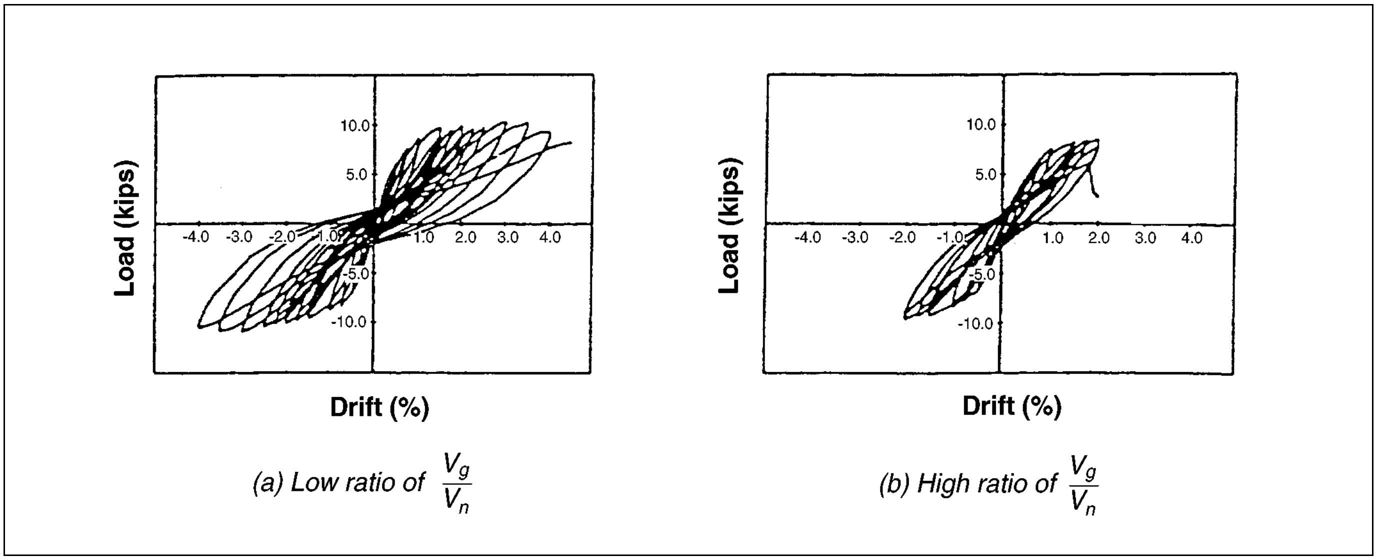


Figure C6-11 Sample Load-Deformation Relations for Reinforced Concrete Slab-Column Connections

function of flexural, shear, torsion, and bond actions. Some detailed models have been reported in the literature (Hawkins, 1980; Luo et al., 1994). A simplified approach, described here, is to model the slab-column frame using the equivalent frame of Figure C6-10c. The column is modeled as described in Section C6.5.2.2B. The slab is modeled according to the general procedures of Section C6.5.2.2B, with initial stiffness according to Section C6.5.4.2A and plastic hinge rotation capacity according to Table 6-14. The connection element between slab and column is modeled as an elasto-plastic component (moderate strain hardening is acceptable) with ultimate rotation capacity according to Table 6-14.

C. Nonlinear Dynamic Procedure

See Section C6.5.2.2C.

Figure C6-11 presents some typical load-deformation relations measured during laboratory tests of slab-column connections where the column did not yield. These illustrate a range of performances that might be anticipated.

C6.5.4.3 Design Strengths

See Section C6.5.2.3 for general discussion on strength of moment frames.

Current technology does not provide accurate strength estimates for slab-column frames. This can be a critical shortcoming, as less-ductile failure modes may in fact predominate even though calculations indicate otherwise. The design of critical structures should take this additional uncertainty into account.

Flexural action of a slab connecting to a column is nonuniform, as illustrated in Figure C6-12. Portions of the slab nearest the column yield first, followed by gradual spread of yielding as deformations increase. The actual flexural strength developed in the slab will depend on the degree to which lateral spread of yielding can occur. The recommendation to limit effective width to the column strip is the same as the design requirement of ACI 318, and represents a lower bound to expected flexural strength. In some cases the full width of the slab will yield. If a greater portion of slab yields than is assumed, the demand on the slab-column connection and the columns will be increased. Nonductile failure modes can result. Shear and moment

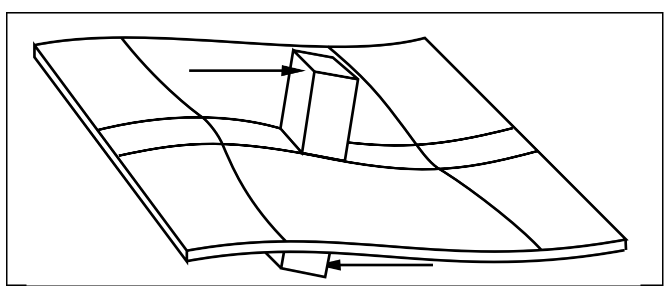


Figure C6-12 Slab Distortion in Flat-Plate Connection under Lateral Load

transfer strength of interior slab-column connections may be calculated using any models that are verified by experimental evidence (Hwang and Moehle, 1993; Hawkins, 1980). It is permissible to use a simplified approached that follows the concepts of *ACI 318-95* (ACI, 1995). According to this approach, connection design strength is the minimum of two calculated strengths. One is the strength corresponding to developing a nominal shear stress capacity on a slab critical section surrounding the column (Figure C6-13). All definitions are according to *ACI 318-95*. In applying this procedure, tests indicate that biaxial moment transfer need not be considered (Pan and Moehle, 1992; Martinez-Cruzado et al., 1991). The second strength corresponds to developing flexural capacity of an effective slab width. The effective width is modified from *ACI 318-95* based on results reported by Hwang and Moehle (1993). Both top and bottom reinforcement are included in the calculated strength.

Shear and moment transfer strength for exterior connections without beams is calculated using the same procedure as specified in *ACI 318-95*. Where spandrel beams exist, the strength should be modified to account for the torsional stiffness and strength of the spandrel beam.

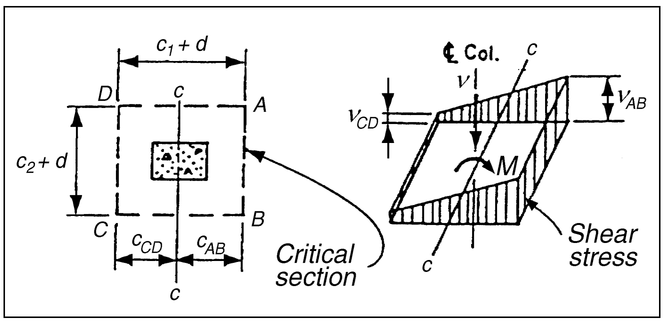


Figure C6-13 Eccentric Shear Stress Model

C6.5.4.4 Acceptance Criteria

A. Linear Static and Dynamic Procedures

For slab-column moment frames, it is preferred that deformation-controlled actions be limited to flexure in slabs, although it may be necessary and acceptable to permit inelastic action in columns and slab-column connections. This preference is partially explained in Section C6.5.2.4. Inelastic response of slab-column connections can be ductile if the level of vertical shear carried from the slab to the column is relatively low (Pan and Moehle, 1989).

Ideally, where the linear procedures of Chapter 3 are used for design, the actions obtained directly from the linear analysis will be used only to determine design values associated with yielding actions in the structure. The design actions in the rest of the structure should be determined using limit analysis procedures considering the gravity forces plus the yielding actions acting on a free body diagram of the component or element. The *Guidelines* specify actions that should be designed on this basis.

Reinforced concrete components whose design forces are less than force capacities can be assumed to satisfy all the performance criteria of the *Guidelines*. However, it is still necessary to check performance of all other components and the structure as a whole.

Slab-column frames with weak columns having widely-spaced transverse reinforcement may be susceptible to story collapse due to column failure. The specified procedure is the same as that specified for beam-column frames in Section 6.5.2.4.

The *m* values were developed from experience and judgment of the project team, guided by available test data (Pan and Moehle, 1989; Martinez-Cruzado et al., 1991; Hwang and Moehle, 1993; Goel and Masri, 1994; Graf and Mehrain, 1992; Meli and Rodriguez, 1979; Durrani et al., 1995).

B. Nonlinear Static and Dynamic Procedures

It is preferred that inelastic response be limited to flexure in beams and columns, or inelastic rotation of slab-column connections. For components whose strength is limited by shear, torsion, and reinforcement development and splicing, the deformability usually is less than for flexure, and stability under repeated deformation cycles is often questionable. Where these latter forms of inelastic action are permitted as part of the design, they should preferably be limited to a few components whose contribution to total lateral load resistance is a minority.

C6.5.4.5 Rehabilitation Measures

The rehabilitation strategies or techniques are similar in principle to those described for beam-column frames in Section 6.5.2.5. The *Commentary* to that section provides general information. In addition, the following aspects apply specifically to slab-column construction.

Jacketing existing slabs, columns, or joints with new steel or reinforced concrete overlays. Where the objective is to improve the strength or ductility of the slab-column connection region, reinforced concrete or steel capitals may be added. These approaches are described by Martinez-Cruzado et al. (1991) and Lou and Durrani (1994). Alternatively, steel plates can be epoxied to both sides of the slab, around the column with through-bolts added to act as plate stiffeners and shear reinforcement (Martinez-Cruzado et al., 1991).

C6.6 Precast Concrete Frames

C6.6.1 Types of Precast Concrete Frames

Many types of precast concrete frames have been constructed since their inception in the 1950s. Some have inherent limited lateral-load-resisting capacity because of the nature of their construction details and because they were consciously designed for wind or earthquake loads. Except for emulated systems and braced systems (Section 6.6.1.1), these frames have capacities to resist lateral loads that are limited by elastic level deformations. In many double tee and single tee systems, as well as others, there is a lack of a complete load path. Brittle welded connections are very common. Many columns and beams lack sufficient confinement steel to provide ductility, and some column systems have inadequate shear capacity as well as base anchorage. Other columns have moment capacity at the base plate that is far beyond their ability to accept the deformations imposed by the global system. Each system may contain details or configuration characteristics that make it unique. Careful study of each unique system is required. In addition, Section C6.12 should be carefully reviewed.

C6.6.2 Precast Concrete Frames that Emulate Cast-in-Place Moment Frames

Frames of this type have been used intermittently since the mid-1950s. Columns with beam stubs are precast with rebar extending from beam or column ends that are connected to other precast members. The joint region has reinforcing extending into it from each of the common members. The joint is “tied” with confining stirrups and then completed by casting the concrete into the gap.

Deficiencies of this type of frame are consistent with those of traditional cast-in-place frames. Additional concerns are for the shear transfer across the joint, confinement of the joint, and tensile steel lap lengths in the joint. The system also requires dowels through the interface between the precast components and the horizontal framing. In many cases this was accomplished using threaded inserts that may or may not have ductile-force-resisting characteristics.

C6.6.3 Precast Concrete Beam-Column Moment Frames Other than Emulated Cast-in-Place Moment Frames

There is a wide variation of frames in this category. The common characteristic is potentially brittle connections that were constructed to resist gravity and wind loads. The addition of shear walls and or steel bracing systems is a primary means for seismically rehabilitating buildings. When employing this or any other approach, a complete load path must be established, with each joint in the system being analyzed for its ability to transmit the required forces and deform appropriate amounts.

C6.6.4 Precast Concrete Frames Not Expected to Resist Lateral Loads Directly

Frames of this category are similar to those of Section C6.6.3, except that it is assumed that other elements resist the lateral loads. Refer to Sections C6.6.3 and C6.6.2.

C6.7 Concrete Frames with Infills

C6.7.1 Types of Concrete Frames with Infills

These types of frames were common starting around the turn of the century. The infill commonly was provided along the perimeter of the building, where it served to clad the building and provide required fire resistance. Design of both the infill and the concrete frame in older buildings typically did not include consideration of the interaction between the frame and infill under lateral loads.

C6.7.1.1 Types of Frames

Infilled frames in older construction almost universally are of cast-in-place construction, and usually are of beam-column construction. However, the general principles of infilled frames as presented in the *Guidelines* are applicable to other types of concrete frames as well. The engineer should anticipate that the frame was designed primarily or exclusively as a gravity-load-carrying frame. The infill probably was not designed to be load-bearing. Frame girders commonly may have been designed without consideration of framing continuity; therefore, only nominal negative moment reinforcement may be present. Beam bottom reinforcement may or may not be continuous into supports. Column longitudinal reinforcement typically was spliced with laps or dowels at or near the floor level. Transverse reinforcement is likely to be relatively light by current standards.

C6.7.1.2 Masonry Infills

No commentary is provided for this section.

C6.7.1.3 Concrete Infills

Concrete infills in existing construction commonly are of cast-in-place concrete. Concrete was used as the infill because of lower cost and because the architectural requirements did not mandate masonry. Concrete infills may be mixed with masonry infills, the concrete infills being used in less visible bays of the framing. The concrete infill in existing buildings commonly was about eight inches thick. Most walls contain some reinforcement, but it may be as light as 3/8-inch bars at 24 inches on centers in one layer in each principal direction. Reinforcement may not extend into the surrounding frame, resulting in a plane of weakness around the perimeter of the infill. Infills may vary over building height, resulting in structural irregularities.

C6.7.2 Concrete Frames with Masonry Infills

C6.7.2.1 General Considerations

This section is concerned primarily with the overall element model, and the behavior and evaluation of the concrete frame. Behavior and evaluation of the masonry infill is covered in detail in Chapter 7.

Infilled frames have demonstrated relatively good performance, although there are some notable exceptions. Lack of toughness in the reinforced concrete framing elements can be a cause of severe damage and collapse, especially for older construction lacking details to provide ductility and continuity. The analysis model should be able to identify deficiencies in the concrete frame related to interaction with the masonry panels. For relatively undamaged infills, the columns act essentially as tension and compression chords of the infilled frame, with relatively large tension and compression forces possible along a substantial length of the column. Adequacy of splices to resist tension, and adequacy of concrete to sustain potentially large compression strains, needs to be considered. As the masonry infill becomes more heavily damaged, in addition to the action as a boundary element, the columns may be loaded locally by large forces from the masonry panel, with the centroid of those forces being eccentric from the beam-column joints. Severe damage to the columns can result. Details of this interaction are in Chapter 7.

C6.7.2.2 Stiffness for Analysis

Chapter 7 contains details on modeling of infilled frames.

The literature contains numerous reports of simulated earthquake load tests on concrete frames with masonry infills; these may provide insight on behavior and modeling issues. Refer to Abrams and Angel (1993), Altin et al. (1992), Fiorato et al. (1970), Gavrilovic and Sendova (1992), Klingner and Bertero (1976), Schuller et al. (1994), and Zarnic and Tomazevic (1985).

C6.7.2.3 Design Strengths

No commentary is provided for this section.

C6.7.2.4 Acceptance Criteria

The acceptance criteria were developed from experience and judgment of the project team, guided by available test data. The strain limits in Table 6-15 are based on experience with axially-loaded columns.

For columns in compression, confinement enables the concrete to sustain load for strains well beyond the crushing strain of 0.002 to 0.003. Ultimate limits for confined columns in compression may be limited by reinforcement buckling. For poorly confined columns, compressive resistance may drop rapidly following initial crushing of concrete. The capacity to sustain gravity loads beyond this point depends on the level of gravity load, and on the capability to redistribute gravity loads to other components, including the masonry infill. Further discussion of compressive strain capacity is provided in Section 6.4.3.

For columns in tension, stress and strain capacity may be limited by the capacity of lap splices. In primary components, failure of a lap splice effectively signals the end of reliable lateral force resistance. In secondary components, splice failure may result in significant loss of lateral load resistance, but gravity load resistance is likely to be sustained; an exception is where axial loads approach the axial load capacity, in which case concrete splitting associated with splice failure may result in reductions in axial compression capacity of the column. Additional discussion on splice strength is provided in Section 6.4.5.

A. Nonlinear Static and Dynamic Procedures

The numerical model should properly represent the load-deformation response of the infilled frame. Figure C6-14 presents some typical load-deformation relations measured in laboratory tests.

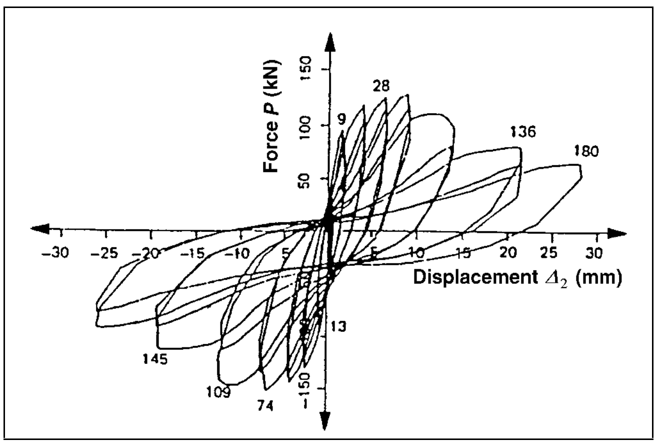


Figure C6-14 Load-Deformation Relation for Masonry-Infilled Reinforced Concrete Frame

C6.7.2.5 Rehabilitation Measures

In addition to the specific procedures listed in this section, the engineer should refer to additional procedures for infills in Chapter 7.

- **Jacketing existing beams, columns, or joints with new reinforced concrete, steel, or fiber wrap overlays.** This approach is especially suitable when overlays are placed over the masonry infill to achieve improved strength and toughness. Overlays may include reinforced concrete, fiberglass, carbon fiber, kevlar, or other materials. Examples are provided in Ehsani and Saadatmanesh (1994) and Zarnic et al. (1986). Jacketing of beams, columns, and joints is not likely to be a primary approach to rehabilitation of existing infilled frames, because it is not possible to fully encase beams or columns due to the presence of the infill.

- **Post-tensioning existing beams, columns, or joints using external post-tensioned reinforcement.** Lateral deformations of slender walls may result in significant tension force requirements for boundary columns, which may lead to unacceptable behavior of reinforcement splices. Post-tensioning can be considered as an option for precompressing columns to avoid excessive tension forces. When this approach is adopted, the design needs to also consider the possible negative effects on column behavior when the lateral forces reverse and the column becomes loaded in compression.

- **Modifying of the element by selective material removal from the existing element.** This is a primary method of rehabilitating existing infilled frames. In general, removal of existing infills should not result in vertical or plan irregularities in the structural system.

- **Improving of deficient existing reinforcement details.** This approach may be useful for improving tension lap strength of existing column lap splices. When this option is selected, chipping of concrete cover may be required; care should be exercised to ensure that core concrete, and bond with existing transverse reinforcement, are not damaged excessively.

- **Changing the building system to reduce the demands on the existing element.** This is a primary method of rehabilitating existing infilled frames. By adding sufficiently stiff elements, it may be possible to reduce design demands on the infills to acceptable levels. Concrete walls may be particularly suitable for this purpose; steel braced frames, and especially eccentrically-braced frames, may lack adequate stiffness to protect the infill from damage. Where new elements are added, the design must ensure adequate connections with adjacent elements. Seismic isolation and supplemental damping may also be used to reduce demands to acceptable levels.

C6.7.3 Concrete Frames with Concrete Infills

C6.7.3.1 General Considerations

Traditionally, a variety of analysis models have been used to model concrete frames with concrete infills. One approach has been to assume that the frame is sufficiently flexible and weak that framing action does not appreciably influence behavior. In this extreme, the frame with infill is modeled as a solid shear wall. This approach is often suitable in cases where the frame is relatively flexible, but may not be suitable for walls with openings, or for stiff frames (typically those with deep spandrels and short columns). Another extreme has been to completely ignore the infill in the numerical model. This approach is often unsuitable because it overlooks potentially significant interaction effects. These effects include overall element strength and stiffness, as well as potentially detrimental effects on columns acting as boundary elements or otherwise interacting with the frame. Detailed discussion of this interaction is provided in Section 6.7.2 and Chapter 7. Braced-frame analogies may be used to identify some aspects of the interaction.

The current state of knowledge does not justify recommendation of generally applicable modeling rules. Engineering judgment provides the only rule of general application. Engineering judgment may be guided by detailed finite-element solutions of subassemblies. Experimental data are lacking; therefore, testing of subassemblies is encouraged where feasible.

C6.7.3.2 Stiffness for Analysis

Because of the lack of experimental data, engineering judgment is required when establishing modeling parameters. Where the frames are relatively flexible and weak, and the infills are in good condition and adequately connected with the frame, the general procedures for walls in Sections 6.8 and 6.9 may provide guidance. Where the frames are relatively stiff and strong, and the infills are relatively weak, the general procedures for concrete frames with masonry infills in Section 6.7.2 may provide guidance.

C6.7.3.3 Design Strengths

Shear strength provided by a concrete infill is likely to depend on the shear strength of the infill itself, and the interface between the infill and the surrounding frame. In existing construction, the infill reinforcement is likely to be relatively light, and is likely to not be anchored into the surrounding frame. As noted in Section 6.8.2.3, where the reinforcement ratio is low, the shear strength is to be calculated using procedures that differ from those in *ACI 318-95* (ACI, 1995). Where the infill reinforcement is not anchored in the surrounding frame, sliding along the interface may occur during lateral loading. In this case, shear is introduced to the frame primarily by direct bearing (lug action) between the infill and the surrounding frame. In this case, shear strength may be limited by direct shear strength of the infill, by local crushing of the infill where it bears against the surrounding frame, or by shear failure of the surrounding frame because of the eccentric bearing of the infill against the frame. These basic behaviors are similar to those described for masonry infills in Chapter 7. Lacking experimental data, the *Guidelines* assume the strength to be limited by direct shear strength of the infill.

Similarly, flexural strength of an infilled frame is likely to be influenced by continuity of the longitudinal reinforcement. Lap splices in the boundary columns may limit strength and deformation capacity. If the infill reinforcement is not anchored in the surrounding frame, it should not be included in the design strength.

C6.7.3.4 Acceptance Criteria

Engineering judgment is required in establishing the acceptance criteria because of the lack of relevant test data. In general, the following aspects should be considered.

- The surrounding frame should be checked for action in tension and compression as described in Section 6.7.2.4. Where portions of the frame are not infilled, the relevant criteria of Section 6.5 should be checked.
- The infilled frame should be checked according to criteria in Section 6.7.2.4.
- Where the relative stiffnesses and strengths of the frame and infill result in effectively composite action, the relevant criteria of Sections 6.8 and 6.9 should be considered.

C6.7.3.5 Rehabilitation Measures

Tests on walls thickened by jacketing have been reported by Goto and Adachi (1987) and Motooka et al. (1984). Infills have also been used to retrofit existing frame construction. Relevant test data on frames rehabilitated by concrete infills can be found in Aoyama et al. (1984), Hayashi et al. (1980), Jirsa and Kreger (1989), and Kahn and Hanson (1979).

C6.8 Concrete Shear Walls

C6.8.1 Types of Concrete Shear Walls and Associated Components

Due to their high initial stiffness and lateral load capacity, shear walls are an ideal choice for a lateral-load-resisting system in a reinforced concrete (RC) structure. Slender walls will normally exhibit stable ductile flexural response under severe lateral loading, but squat walls are more likely to be governed by shear response, so they must be designed for lower ductilities. In residential construction, the generous use of walls provides ample redundancy and load capacity to keep seismic forces and deformation demands relatively low. However, due to architectural restraints in office buildings, there tend to be fewer shear walls, and horizontal spans are kept as short as possible. Thus, these walls are usually slender, and seismic deformation demands tend to be high.

There are three general structural classifications in which shear walls are used as the primary lateral-load-resisting elements. In bearing wall systems, shear walls serve as the primary members for both gravity and lateral load resistance. Such structures have often been considered to behave in a nonductile manner when subjected to large lateral loads, but studies of several bearing wall buildings following the 1985 Chile earthquake have shown that such structures may be very reliable for seismic resistance if there is a high percentage of wall area to total floor area (Wood et al., 1987; Sozen, 1989; Wood, 1991b; Wallace and Moehle, 1992 and 1993; Wight et al., 1996).

When a shear wall is assumed to be the only lateral-load-resisting system and a space frame is provided to carry most of the gravity load, the resulting structural system is commonly referred to as a shear wall system. In such systems the shear walls often form the perimeter of an interior core that contains the elevator shaft and stairways. In some cases the core walls will work in combination with isolated walls that are distributed around the perimeter of the building, to increase the torsional stiffness of the building.

Where shear walls are combined with a space frame that carries most of the gravity load and also assists in resisting lateral loads, the structure is referred to as a dual (wall-frame) system. Again, the most common use for the shear walls in such a system would be to form an interior core. Because of the different elastic displacement modes for walls and frames, the dual system offers significant stiffness benefits in the elastic range of lateral loading. For inelastic lateral loading, the frame offers a second line of defense, which provides significant lateral stiffness and strength after initial yielding at the base of the shear walls.

For any one of these three general structural systems, shear walls that are in the same plane may be joined together at each floor level with coupling beams to form a coupled-wall system. As with a wall-frame system, the coupled-wall system offers a significant increase in lateral stiffness compared to the algebraic sum of lateral stiffnesses of isolated shear walls. Under inelastic lateral loadings, the coupling beams can provide significant energy absorbtion if they are properly detailed.

In bearing wall systems, the shear walls may have a pattern of large openings in both the horizontal and vertical directions. Such walls are commonly referred to as either a “framed-wall” or a “perforated-wall system.” Perforated walls are typically used along the exterior of buildings to form a repetitive pattern of window openings. The behavior of such a wall system is more often dominated by the behavior of the individual vertical and horizontal wall segments, than by the overall proportions of the wall. The vertical wall elements are commonly referred to as “wall piers.” There is no common terminology for the horizontal wall segments that resemble deep beams. For all the tables presented in the *Guidelines*, the term “wall segments” refers to both the horizontal and vertical members of a perforated wall.

Although they are frame elements, coupling beams and columns that support discontinuous shear walls are included in this section of the *Guidelines*. When these elements are used in a shear wall system, they will commonly have large ductility demands under large lateral load reversals. Therefore, the detailing of the reinforcement in such members, particularly the transverse reinforcement, is critical to the behavior of these elements. Of course, the inelastic behavior of these elements will strongly influence the lateral load response of the shear wall system in which they are located.

C6.8.1.1 Monolithic Reinforced Concrete Shear Walls and Wall Segments

A slender shear wall will commonly have longitudinal reinforcement concentrated either along its horizontal edges or within a boundary element. For both cases, the percentage of longitudinal steel concentrated at the wall edge and the amount and spacing of the transverse reinforcement used to confine that steel will have a significant influence on the inelastic lateral load response of the shear wall. A large percentage of longitudinal reinforcement will increase the shear required to cause flexural yielding under lateral loading, and will increase the compressive strains along the compression edge of the wall. The increased shear could either trigger an early shear failure or cause a more rapid deterioration of stiffness under lateral load reversals. The high compression strains could lead to concrete crushing and rebar buckling unless the compression edge of the wall is well confined by transverse reinforcement.

Squat shear walls normally have a uniform distribution of vertical and longitudinal steel. If the percentage of vertical steel is low, flexural behavior may govern inelastic response under lateral loads. If shear governs the lateral load behavior, either the available shear ductility should be assumed to be a small value, or the shear strength of the wall should be designated as a force-controlled action. Details are given in Section 6.8.2.4 of the *Guidelines*.

C6.8.1.2 Reinforced Concrete Columns Supporting Discontinuous Shear Walls

RC columns that support discontinuous shear walls are subjected to large force and displacement demands during severe ground shaking. The damage inflicted on the first story columns of the Olive View Hospital during the 1971 San Fernando earthquake is an often-used example of the demands placed on such columns. Modern building codes have limitations on stiffness discontinuites that tend to eliminate this type of construction. However, there are existing RC buildings with shear walls that are not continuous to the foundation level. For these buildings, the columns that support the discontinuous walls will need to be carefully analyzed.

In most cases, the shear strength of columns supporting discontinuous shear walls will be a force-controlled action. These columns should be analyzed as displacement-controlled members only if they have transverse reinforcement that satisfies ductile detailing requirements of modern codes. Even in these cases, the permitted ductility values will be very low. Following the 1995 earthquake in Kobe, Japan, there have been reports (Bertero et al., 1995; Watabe, 1995) of damage to RC columns supporting discontinuous shear walls in very modern RC structures. The columns were well detailed, but the displacement demands were excessive.

C6.8.1.3 Reinforced Concrete Coupling Beams

RC coupling beams are normally deep with respect to their span. Observations of post-earthquake damage to concrete shear wall buildings have repeatedly shown diagonal tension failures (severe X-cracking) in coupling beams. The most common cause of this damage is insufficient shear strength to develop the beam's flexural strength under repeated cyclic loading. Any contribution from the concrete to the shear capacity should be ignored and closed stirrups should be provided at a close spacing (ð d/4). However, even these measures may only delay, and will not necessarily prevent, an eventual shear failure under repeated large load reversals (Paulay, 1971a).

Research (Paulay, 1971b) has shown that coupling beams designed with primary reinforcement arranged in a diagonal pattern over the length of the beam will exhibit more stable behavior under large load reversals than will conventionally reinforced coupling beams. When diagonal reinforcement is used, it should be designed to resist the vertical shear forces that accompany flexural yielding of the reinforcement.

C6.8.2 Reinforced Concrete Shear Walls, Wall Segments, Coupling Beams, and RC Columns Supporting Discontinuous Shear Walls

C6.8.2.1 General Modeling Considerations

Using equivalent beam-column elements to model the elastic and inelastic response of slender shear walls is a fairly common practice. A primary reason for using equivalent beam-column models for shear walls is because numerous frame analysis programs are available to a structural engineer. The use of an equivalent beam-column model to represent inelastic behavior of shear walls and wall segments is normally acceptable for slender elements with aspect ratios above those stated in the *Guidelines*, where flexural response will dominate. However, in all these cases the equivalent beam-column must incorporate shear deformations and the beams connecting to the equivalent beam-column element must have long rigid end zones to properly simulate the horizontal dimension of the shear wall. Results from a large number of shear wall tests have been summarized by Wood (1991a).

For squat shear walls, or other walls where shear deformations will be significant, a more sophisticated wall model should be used. This model should incorporate both elastic and inelastic shear deformations, as well as the full range of flexural behavior. Researchers have suggested the use of multiple spring models (Otani, 1980; Otani et al., 1985; Alama and Wight, 1992), and multi-node link models to represent an RC shear wall (Charney, 1991).

Most coupling beams have small span-to-depth ratios, so any beam element used to model a coupling beam must incorporate shear deformations. Several researchers have developed special beam elements specifically for simulating the response of an RC coupling beam (Saatcioglu, 1991).

Columns that support discontinuous shear walls can be modeled with a beam-column element similar to that used in most frame analysis programs. However, the element should include shear deformations and the possible rapid loss of shear strength under large lateral deformations and high axial load.

C6.8.2.2 Stiffness for Analysis

Typical sources of flexibility in RC members were discussed in Section C6.4.1.2.

A. Linear Static and Dynamic Procedures

The linear procedures of Chapter 3 assume that the element stiffness used in analysis approximates the stiffness of that element at displacement amplitudes near its effective yield displacement. At such displacement levels, the effective element stiffness will be significantly less than the gross stiffness commonly used in conventional design practice. A discussion of how the effective stiffness may vary as a function of the source of deformation and level of stress is given in Section C6.4.1.2. In lieu of a more precise analysis, the effective element stiffnesses for linear procedures should be based on the approximate values given in Table 6-4.

B. Nonlinear Static and Dynamic Procedures

The nonlinear procedures of Chapter 3 require the definition of the typical nonlinear load-deformation relationship for each displacement-controlled action. For the NSP, it is sufficient to define a load-deformation relationship that describes the behavior of an element under monotonically increasing lateral deformations. For the NDP, the same basic load-deformation relationship can be used as a backbone curve, but it is also necessary to define rules for the load-deformation relationship under multiple reversed deformation cycles. Figure 6-1 shows typical load-deformation relationships that may be used for the NSP. Definitions of the key points in this figure are given in the *Guidelines*.

When using the basic load-deformation curves given in Figure 6-1, the ordinates (loads) are to be a function of the member strengths defined in Section 6.8.2.3. The deformation values (x-axis) are to be defined as either plastic hinge rotations, drifts, or chord rotations, depending on the type of element involved and whether the element's inelastic response is governed by flexure or shear. Plastic hinge rotations are used where flexure governs the inelastic response for shear walls and wall segments, and for RC columns supporting discontinuous shear walls. It should be clear that RC columns that have shear strengths below the shear required to develop flexural hinging are not included in this discussion.

A sketch of the first story of a deformed shear wall governed by flexure is given in Figure C6-15. The length of a plastic hinging region in an RC member is typically defined to be somewhere between 0.5 and 1.0 times the effective flexural depth of the member. For RC members where shear deformations are significant, the plastic hinging length tends toward the upper end of this range, and vice versa. Therefore, for the shear walls the plastic hinging region will extend very close to, if not beyond, one story height of the member. In these cases it is appropriate to limit the length of the plastic hinging zone to one story height. For wall segments that often have small length-to-depth ratios, the plastic hinging zone may extend to mid-length of the member. For those cases, the length of the plastic hinging zone is limited to one-half the length of the member. For RC columns that support discontinuous shear walls, the plastic hinge length is taken to be taken as one-half the effective flexural depth, as is done for typical RC frame members.

For members whose inelastic response is controlled by shear, Figure 6-1(b) should be used to characterize the inelastic behavior of the member and *drift* should be used as the deformation value. Drift for shear walls is defined as the lateral displacement over one story height, divided by story height (Figure C6-16). For wall segments, drift is defined as the transverse displacement of the member over its length, divided by the member length.

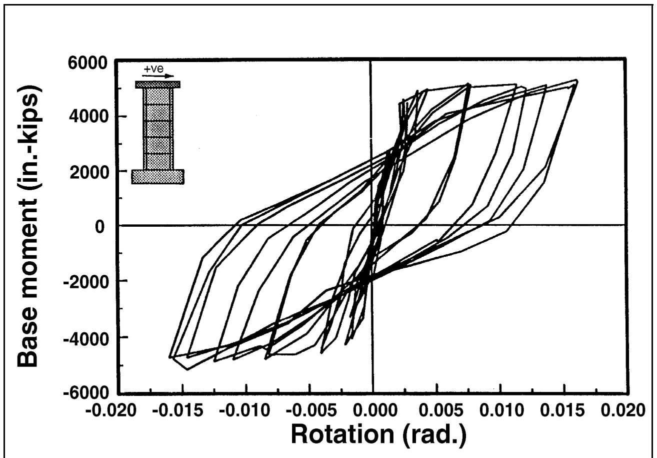


Figure C6-15 Shear Wall Base Moment versus First-Story Rotation Relationship (Specimen W-1, Ali and Wight, 1991)

Figure 6-1(b) is also used to characterize the inelastic behavior of coupling beams, whether their inelastic response is governed by flexure or by shear. Chord rotation, as defined in Figure 6-4, is considered to be the most appropriate deformation measure for inelastic response of coupling beams.

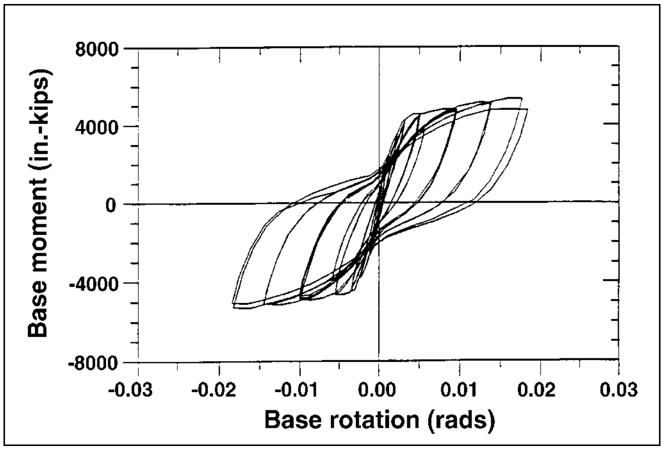


Figure C6-16 Shear Wall Base Moment versus Base Rotation Relationship (Specimen RW1, Thomsen and Wallace, 1995)

Values for the hinge rotation values *a* and *b* (which are described in Figure 6-1(a) and given in Table 6-17) and the drift or chord rotation values *d* and *e* (which are described in Figure 6-1(b) and given in Tables 6-17 and 6-18) are based on experimentally observed behavior of RC members and the engineering judgment of the project team. Experimental results of the inelastic behavior of elements defined in Tables 6-17 and 6-18 are described in the following sections of the *Commentary.*

C6.8.2.3 Design Strengths

Component strengths are to be calculated based on the principles and procedures from *ACI 318-95* (ACI, 1995) and the 1994 *NEHRP Recommended Provisions* (BSSC, 1995), with some modification to reflect different purposes of the *Guidelines* and those documents. The design engineer must consider all potential failure modes that may occur at any section along the length of the member under consideration.

When calculating the nominal flexural yield strength of a shear wall or wall segments, it is assumed that only the longitudinal steel in the outer portions of the wall will yield initially. As lateral deformations increase, section rotations in the plastic hinging region will increase to the point that essentially all the longitudinal steel will be yielding. This point is assumed to represent the nominal flexural strength of the member. For both the yield strength and nominal flexural strength calculation, the value for the yield strength of the reinforcement should be increased by 25% to account for actual yield strengths exceeding the specified yield strength, and the onset of strain hardening in the reinforcement at rotations beyond the yield rotation.

For shear-controlled shear walls and wall segments, no difference is assumed between the shear yield strength and the nominal shear strength of the element. Also, the reinforcement strength is set equal to the specified yield strength. These conservative assumptions are used for additional safety because shear-controlled members have less ductility and are usually more brittle that flexure-controlled members.

Similar procedures are used to evaluate the nominal flexure and shear strengths of coupling beam elements. For RC columns supporting discontinuous shear walls, nominal strengths are based on the procedures developed in Section 6.5.2.3 of the *Guidelines*.

C6.8.2.4 Acceptance Criteria

A. Linear Static and Dynamic Procedures

The acceptance criteria of Chapter 3 require that all component actions be classified as either displacement-controlled or force-controlled actions. For most RC members, it is preferable that deformation-controlled actions be limited to those members where flexural actions govern the nonlinear response. However, for some of the RC members covered in this section, shear may govern the strength and nonlinear response. Therefore, Table 6-20 includes *m* values for members controlled by shear. For RC columns that support discontinuous shear walls, *m* values are only given for members governed by flexure. Shear-critical RC columns must be considered as force-controlled components.

Where the linear procedures of Chapter 3 are used for design, they should be restricted to determining design values for yielding parts of the structure. The design actions for the force-controlled portions of the structure should be determined by statics considering the gravity forces plus the yielding actions for the deformation-controlled components in the structure. Members whose design forces are less than their respective capacities can be assumed to satisfy all the performance criteria of the *Guidelines*.

One example of laboratory data used to determine *m* values is given in Figure C6-15 (Ali and Wight, 1991). The figure shows the base moment versus base rotation relationship for a one-fifth scale five-story shear wall specimen. The specimen generally satisfies the conditions listed in the first row of Table 6-19. The wall reinforcement was symmetrical and the axial load was approximately equal to $0.1 t_w l_w \sqrt{f'_c}$. The wall had confined boundary elements and the maximum shear stress recorded during the test was approximately $3\sqrt{f'_c}$. A single lateral load was applied at the top of the specimen, so the base moment in Figure C6-15 was the lateral load multiplied by the height of the specimen. The base rotation was measured over one story height, which was approximately 0.55 times the length of the wall.

The general results given in Figure C6-15 indicate that this specimen was able to achieve base rotations exceeding 0.015 radians without any loss of strength. Clearly, the specimen could have achieved higher base rotations, but the testing was terminated because the maximum displacement capacity of the testing equipment had been reached. Although the interpretation of the yield point is somewhat subjective, it appears that the base rotation at yield for this specimen was approximately equal to 0.0025 radians. Thus, this specimen achieved a base rotational ductility of 6.0, without any indication of strength deterioration.

Similar test results have been reported by other researchers (Thomsen and Wallace, 1995; Paulay, 1986) for shear walls that also generally fit the conditions listed in the first row of Table 6-19. The base moment versus base rotation results from Thomsen and Wallace are shown in Figure C6-16, and the lateral load versus top lateral displacement results from Paulay are shown in Figure C6-17. The results in Figure C6-16 are remarkably similar to those in Figure C6-15, and actually indicate a maximum base rotation approaching 0.020 radians. The results given in Figures C6-15 and C6-16 were used to determine appropriate *m* values for the first row of Table 6-19. The test results from Paulay are presented as further confirmation of the available ductility in shear walls satisfying the listed conditions.

Two other sets of test results from Thomsen and Wallace for shear walls governed by flexure are given in Figures C6-18 and C6-19. Both of these results are for walls with T-shaped cross sections. Positive moment corresponds to putting the flange of the section into compression, and negative moment corresponds to putting the stem of the section into compression. Before conducting these tests, Thomsen and Wallace had done analytical studies of T-shaped cross sections (Figure C6-20). The results of those studies had clearly

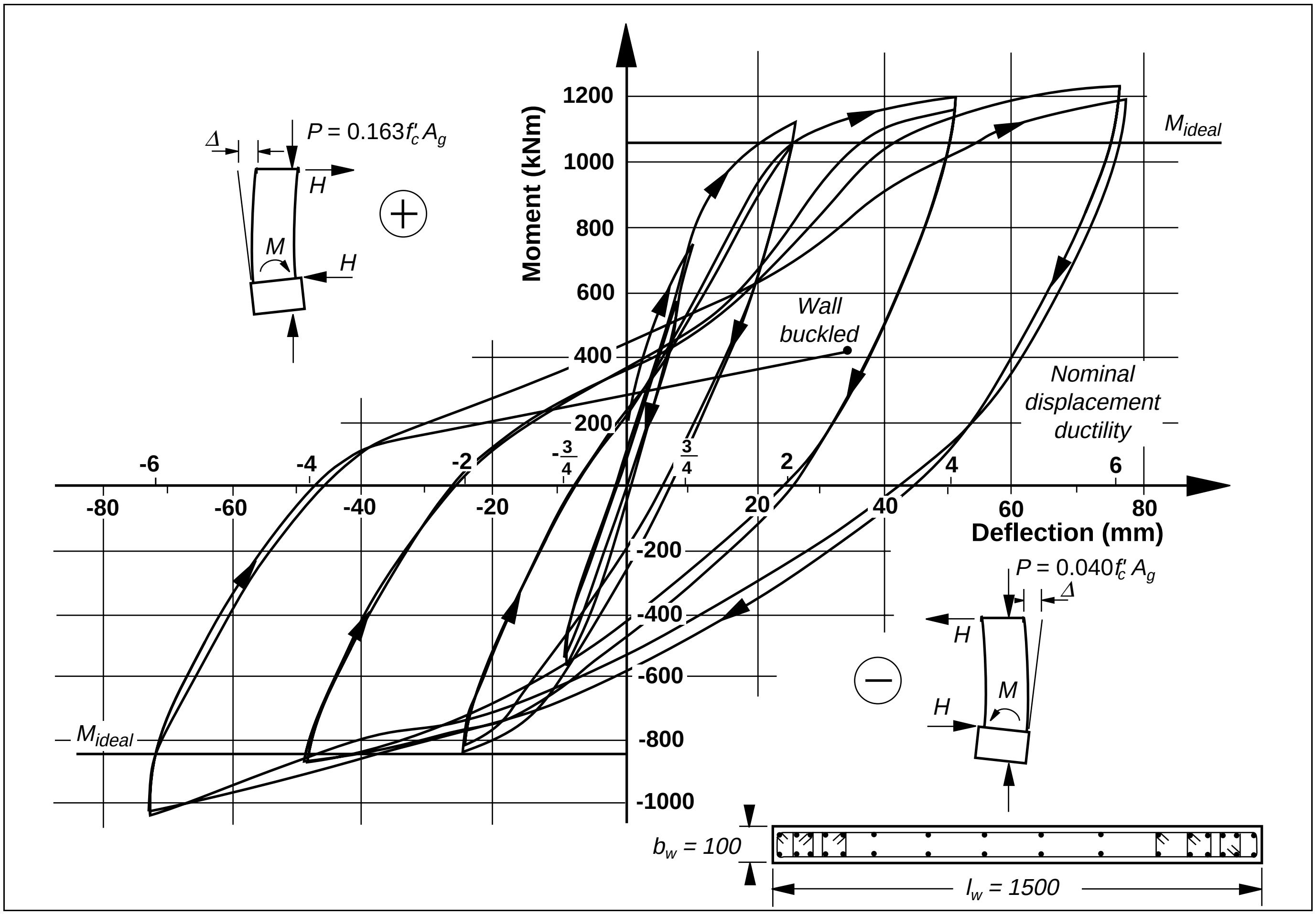


Figure C6-17 Lateral Load versus Top Displacement Relationship (Paulay, 1986)

indicated that less ductility should be expected when the stem of the T-section is subjected to compression.

The results shown in Figures C6-18 and C6-19 for negative bending should correspond to the conditions represented by rows three and seven, respectively, of Table 6-19. The axial load acting on the specimens was low, but the large difference between the tension steel area from the flange versus the compression steel area from the stem put the coefficient for the parameter in the first column of Table 6-19 above the given limit of 0.25. The results in Figure C6-18 represent a well-confined boundary region, and those in Figure C6-19 represent a poorly confined boundary region. The shear stress in both specimens was below $3\sqrt{f'_c}$.

The specimen shown in Figure C6-18 demonstrates a reasonable amount of ductility and reaches a maximum

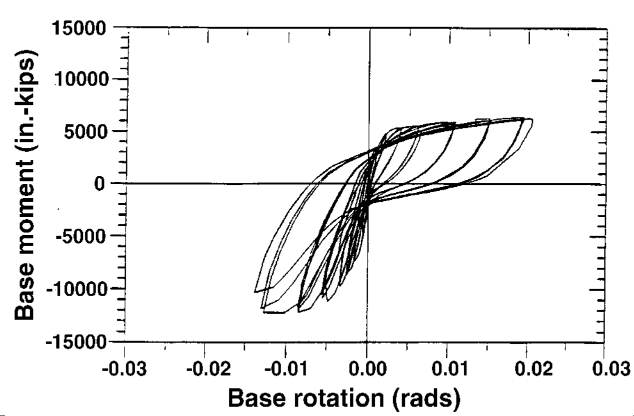


Figure C6-18 Shear Wall Base Moment versus Base Rotation Relationship (Specimen TW2, Thomsen and Wallace, 1995)

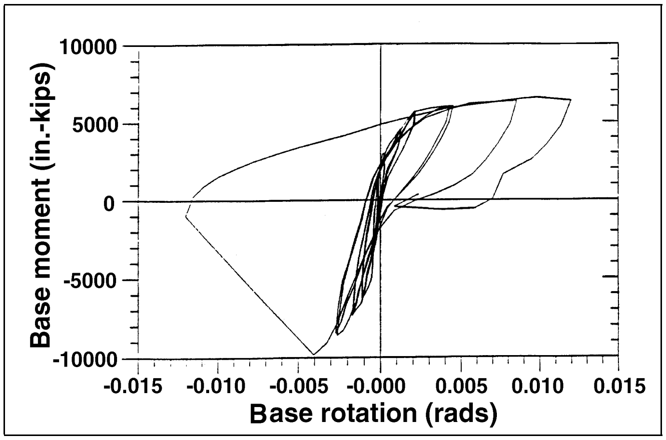


Figure C6-19 Shear Wall Base Moment versus Base Rotation Relationship (Specimen TW1, Thomsen and Wallace, 1995)

base rotation of approximately 0.013 before experiencing a substantial loss in capacity. Because this specimen has less ductility and a more rapid strength loss at higher rotation values than shown by the specimens in Figures C6-15 and C6-16, lower *m* values are used in the third row of Table 6-19. The specimen in Figure C6-19 shows a very low amount of ductility, and this result is reflected in the *m* values used in the seventh row of Table 6-19.

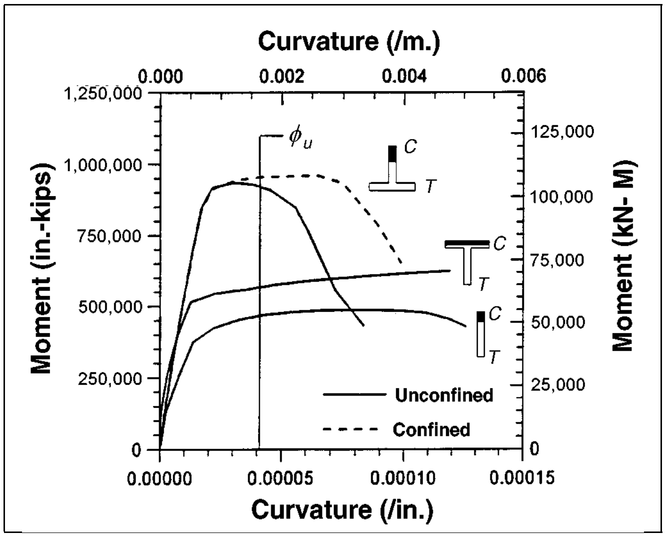


Figure C6-20 Analytical Moment-Curvature Relationship for Rectangular and T-Shaped Wall Sections (Thomsen and Wallace, 1995)

Design engineers must use some judgment when interpreting test results for isolated specimens similar to those shown in Figure C6-19. When the compression zone of this specimen becomes unstable, the specimen fails immediately because there is nowhere else for the load to go. However, if this wall were contained within a building structure consisting of several walls and columns, its response would be much more stable. When the compression zone of this specimen started to deteriorate, it would become much less stiff, and loads in the structure would redistribute to stiffer lateral-load-resisting members. This wall could then be subjected to larger deformations while carrying less load. This assumed behavior is reflected in the *m* values of Table 6-19 and the residual strength values listed in Table 6-17.

Although flexure is the preferred mode of inelastic response for RC members (elements and components), shear will control the inelastic response of certain shear wall, wall segment, and coupling beam elements. Test results for a shear wall controlled by shear are shown in Figure C6-21 (Saatcioglu, 1991). This was a one-story specimen with a height of 1000 mm. Thus, a lateral top deflection of 10 mm corresponds to a 1% story drift.

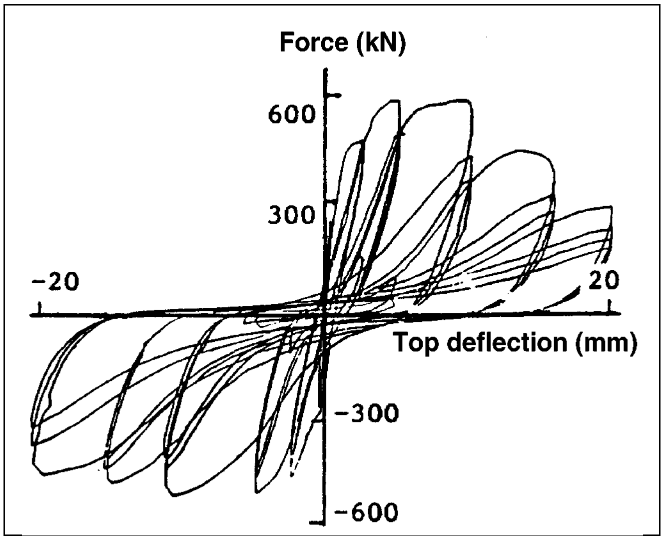


Figure C6-21 Lateral Shear Force versus Top Displacement of Shear Wall Specimen 1 (Saatcioglu, 1995)

As stated previously, the determination of the yield point is somewhat subjective, but could be assumed to occur at a top displacement of approximately 2.5 mm (drift of 0.25%). Beyond this point, the specimen exhibited the development of diagonal cracks that continued to open wider as the lateral displacements were increased. The specimen reached a top displacement of 10 mm (drift of 1.0%) before it experienced a significant deterioration of its shear capacity. The specimen was able to achieve top displacements of 20 mm (drift of 2%) without a dramatic failure.

The results of another shear wall test by Saatcioglu are given in Figure C6-22. This specimen had more shear reinforcement, but suffered a shear sliding failure along its base. Although the hysteresis loops are much more pinched than those shown in Figure C6-21, the specimen still has significant deformation capacity without experiencing a sudden failure.

Again, judgment must be used with these test results to determine the *m* values given in the first row of Table 6-20 and the residual capacity given in the first row of Table 6-18.

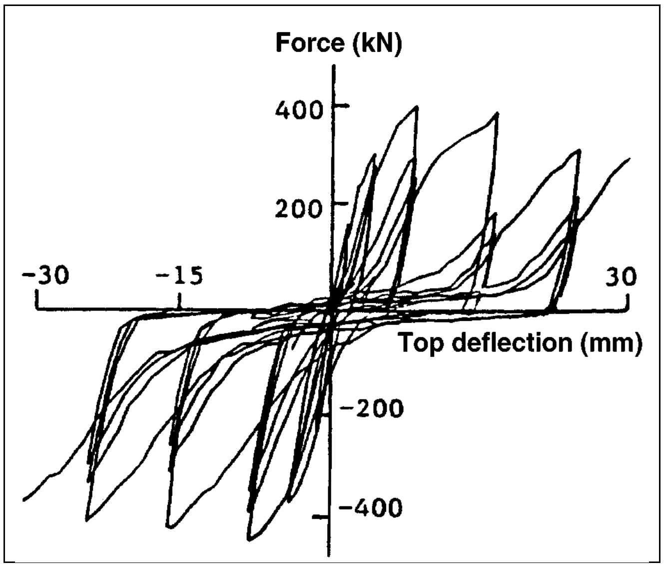


Figure C6-22 Lateral Shear Force versus Top Displacement of Shear Wall Specimen 4 (Saatcioglu, 1995)

Coupling beams are another RC element whose inelastic response is often controlled by shear. Measured lateral load versus chord rotation results for RC coupling beam specimens tested in New Zealand (Paulay, 1971a and 1971b) are presented in Figure C6-23. Both specimens had conventional longitudinal reinforcement and carried maximum shear stresses that exceeded $6\sqrt{f_c'}$. The results for a specimen with conforming transverse reinforcement are given in Figure C6-23; the results for a specimen with nonconforming transverse reinforcement are given in Figure C6-24. Thus, these results should correspond to conditions given in the second and fourth rows, respectively, of Part ii of Table 6-20.

The results shown in Figure C6-23 indicate that the specimen was subjected to only one load reversal after the yield capacity of the specimen was achieved. Thus, the test results are more of a monotonic backbone type curve. However, some required information can be obtained from these results. If one assumes that yield occurred at a chord rotation of approximately 0.004 radians, it then appears that the specimen achieved a rotational ductility of approximately three in each direction. The amount of strength deterioration that would have occurred at this ductility level cannot be determined because of the vary large displacement excursion in the negative direction. However, that large excursion does indicate that rotational ductilities as large as four are possible with little or no loss in capacity for monotonic loading.

Test results for the specimen with nonconforming transverse reinforcement are shown in Figure C6-24. Although the scale for the chord rotation axis has been expanded, it is clear that this specimen had a lower stiffness and a more pinched hysteretic response than was obtained for the specimen that had conforming transverse reinforcement. Thus, the *m* values that are given in the fourth row of Part ii of Table 6-20 are reduced from those given in the second row of Part ii.

A third set of test results from same series of RC coupling beam tests is given in Figure C6-25. This specimen's primary reinforcement was diagonal reinforcement, so it corresponds to the last row of Table 6-19. Clearly, the test results for this specimen indicate that larger rotational ductilities can be obtained and that the lateral load versus rotational hysteresis loops are fuller than obtained for the specimens with conventional longitudinal and transverse reinforcement. This improved behavior is reflected in the large *m* values given in the last row of Table 6-19.

B. Nonlinear Static and Dynamic Procedures

Inelastic response is only acceptable for those actions listed in Tables 6-17 and 6-18. Deformations

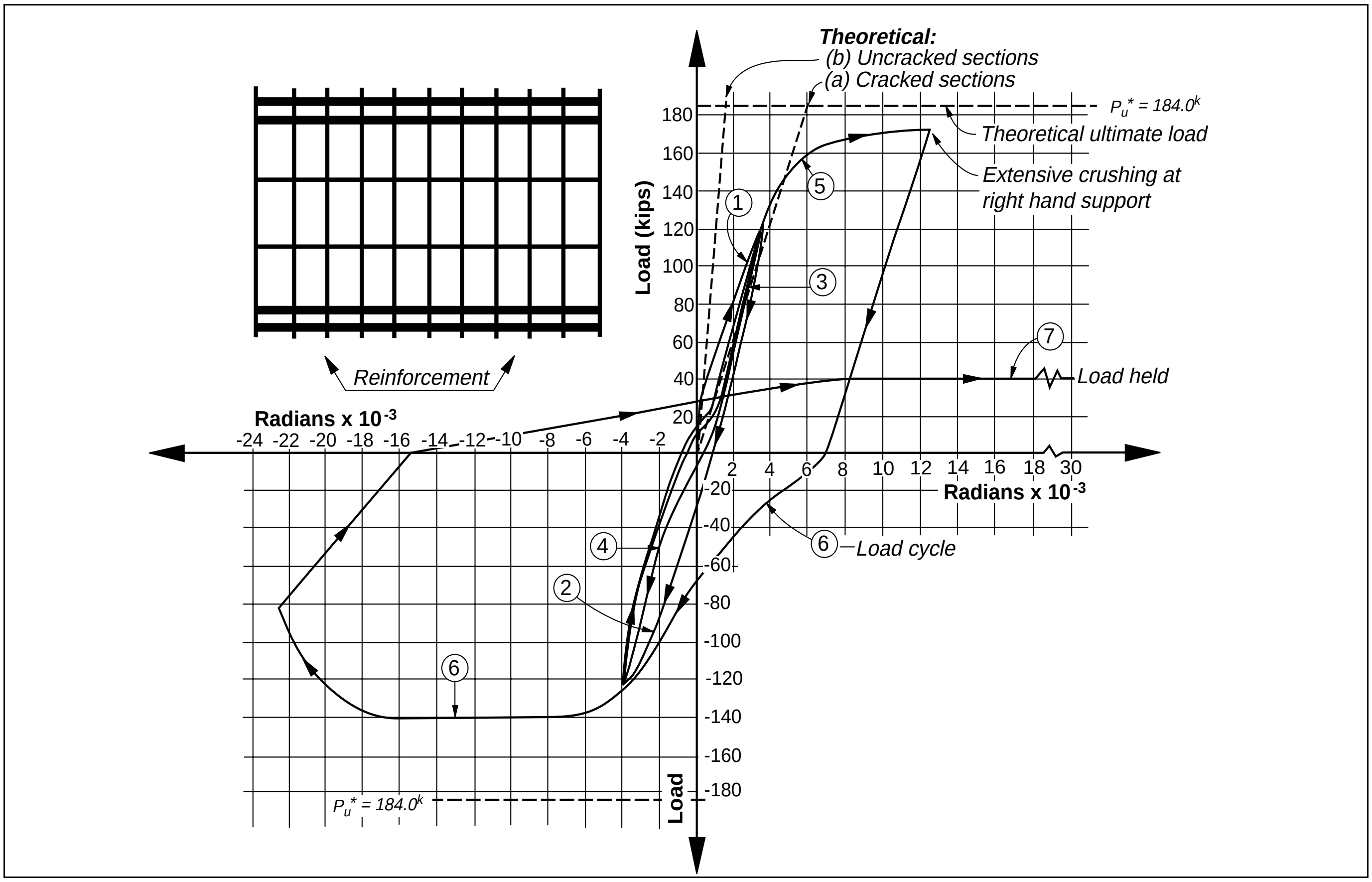


Figure C6-23 Lateral Load versus Chord Rotation Relationship Beam 315 (Paulay, 1971b)

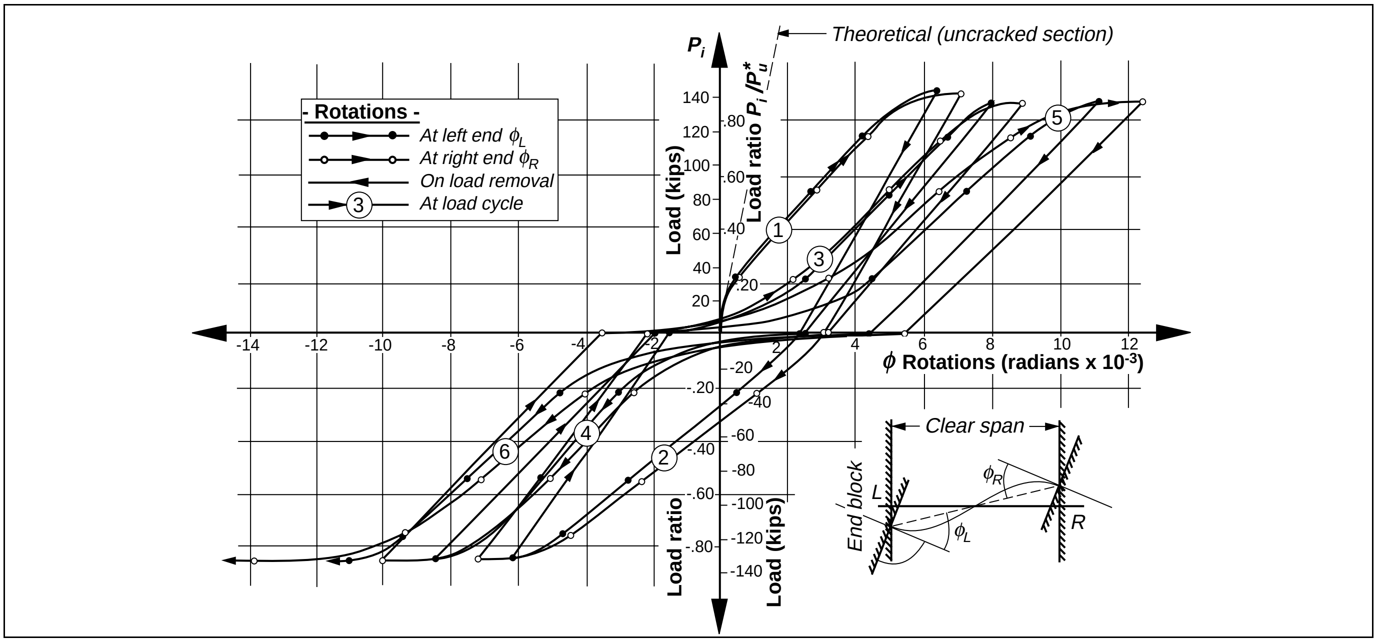


Figure C6-24 Lateral Load versus Chord Rotation Relationship Beam 312 (Paulay, 1971b)

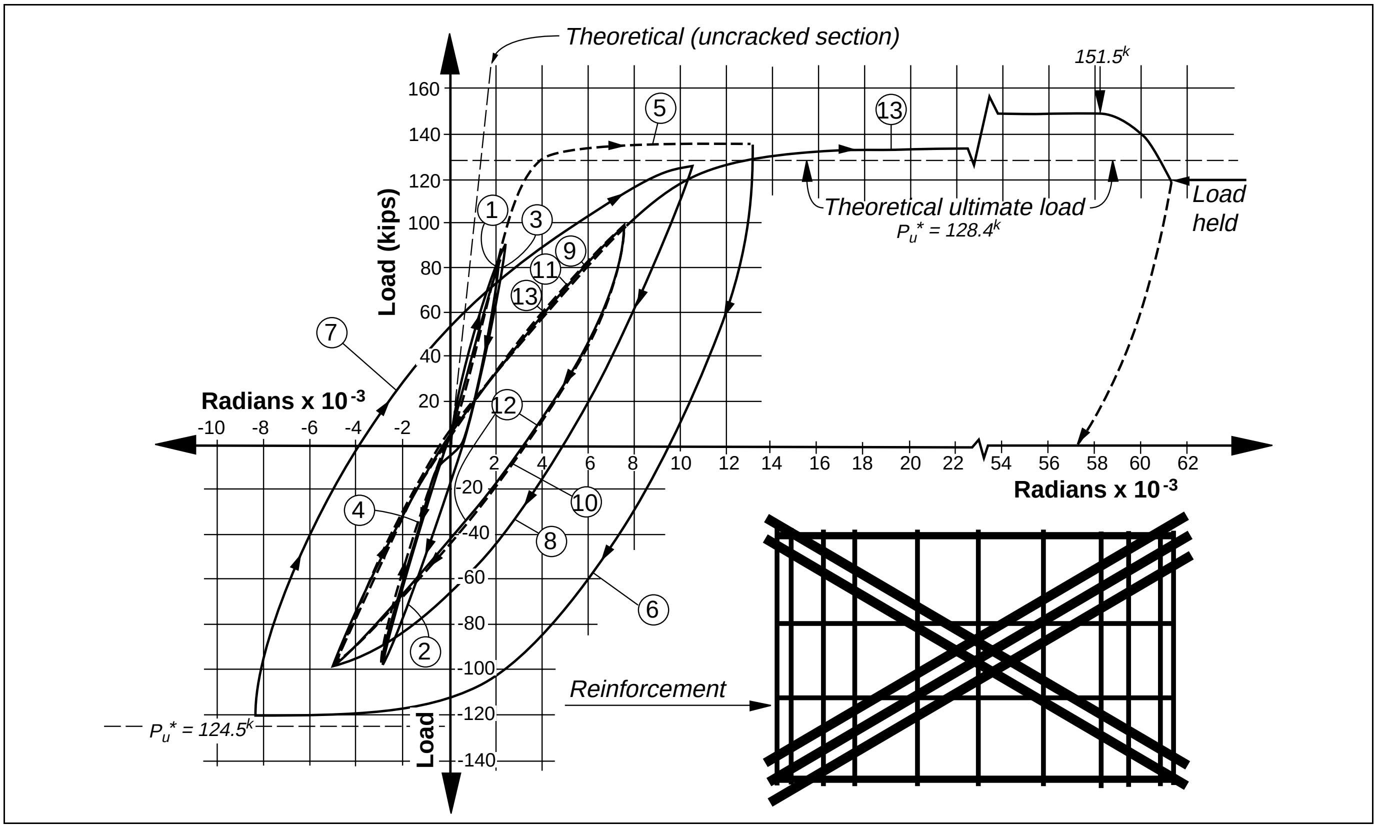


Figure C6-25 Lateral Load versus Chord Rotation Relationship Beam 316 (Paulay, 1971b)

corresponding to these actions shall not exceed the plastic hinge rotation, drift, or chord rotation capacities given in these tables. The deformation values for the nonlinear procedures given in these tables were developed from the experience and judgment of the project team, guided by available test results. The various experimental results referred to in the previous paragraphs are also used here to justify the deformation values given in Table 6-17 and 6-18.

The shear wall test results given in Figures C6-15 and C6-16 correspond to the first row of Table 6-17, and were used to develop the values of *a*, *b*, and *c* required to define the load versus deformation curve given in Figure 6-1(a) of the *Guidelines*. If it is assumed that yielding occurred at a hinge rotation of 0.0025 radians, both specimens reached a plastic rotation (inelastic rotation beyond the yield rotation) of 0.015 radians (value of *a*) without a significant loss in strength.

Because both of the tests referred to here were terminated before the shear wall specimen demonstrated a significant loss in strength, judgment is required to determine what plastic hinge rotations could be reasonably obtained and what residual strength the specimen would have at that deformation state. Both sets of researchers were reporting distress in the wall compression zones at the end of the tests, and the last deformation cycle in Figure C6-17 does show some drop in lateral load capacity. Thus, it was assumed that the plastic rotations could have increased to 0.020 radians (value of *b*), and the specimen could still have carried 75% (value of *c*) of its maximum loads.

The test results shown in Figures C6-18 and C6-19 were used to justify values in the third and seventh rows, respectively, of Table 6-17. Specifically, loading in the negative direction corresponded to the tabulated values. Again, if the yield rotation is taken to be approximately 0.0025 radians, the specimen in Figure C6-18 obtains a plastic rotation of 0.009 radians without an apparent loss in lateral load capacity, and could probably obtain a plastic rotation of 0.012 radians and still maintain 60% of its lateral load capacity. The results in Figure C6-19 indicate that this specimen did not have much deformation capacity beyond yield. However, as noted previously, these test results are for an isolated specimen; the shear wall behavior would be different if the wall was contained within a building structure with several lateral-load-resisting elements. Thus, it was assumed that the wall specimen could have obtained a plastic rotation of 0.003 radians without a significant loss in lateral load capacity. At higher rotations, the load capacity will quickly deteriorate. Thus, at a plastic rotation of 0.005 radians it was assumed that the lateral load capacity would have dropped to 25% of its maximum value.

For shear walls and wall segments controlled by shear, drift was selected as the appropriate deformation parameter, and the *d* and *e* parameters defined in Figure 6-1(b) of the *Guidelines* were selected as the appropriate measures of inelastic deformation.

Test results given in Figure C6-21 are for a shear wall specimen whose inelastic behavior was governed by shear. The web reinforcement ratio used in this specimen was approximately 0.0025, so these results should correspond to the entries in the first row of Table 6-18. Recalling the previous discussion of these test results, a lateral deflection of 10 mm corresponds to a 1.0% story drift. The test results indicate that the specimen could have been cycled at a maximum story drift of 0.75% (*d* value) without a significant loss in lateral load capacity. At the end of the test the specimen was cycled to story drifts of 2.0% (*e* value) and still maintained approximately 40% (*c* value) of its maximum lateral load capacity.

It should be noted that the test results in Figure C6-22 are for a specimen with a large web reinforcement ratio, so the failure of this specimen was due to sliding shear failure at the base of the structure. Thus, it is more appropriate to use the results given in Figure C6-21 for determining the values in Table 6-18.

Chord rotations were selected as the appropriate deformation parameter for shear wall coupling beams, and the backbone curve given in Figure 6-1(b) of the *Guidelines* was used to define the inelastic behavior of coupling beams. The test results shown in Figures C6-23 and C6-24 represent RC coupling beams whose inelastic behavior was governed by shear; these results correspond to rows two and four, respectively, of Part ii of Table 6-18. The results given in Figure C6-23 indicate that the specimen reached chord rotation angles of 0.012 radians (*d* value) in each direction without a significant decrease in lateral load capacity. Not many load cycles were completed for this specimen, but it probably could have maintained at least 30% (*c* value) of its maximum lateral load capacity at chord rotations of 0.020 (*e* value) in each direction.

The results shown in Figure C6-24 indicate that the specimen maintained its lateral load capacity at relatively large chord rotations, but the hysteretic response was very pinched. To account for the low stiffness of this member and its poor hysteresis response, *d* was set equal to 0.008 radians, *e* was set equal to 0.012 radians, and *c* was set equal to 25%.

The lateral load versus chord rotation test results for a shear wall coupling beam with diagonal reinforcement, which corresponds to the conditions for the last row in Table 6-17, are given in Figure C6-25. Again, this specimen was not subjected to many loading cycles, or to large levels of chord rotation, but the given test results indicate a very ductile response that is stable at large chord rotation values. Based on the given test results, *d* was selected to be 0.030 radians, *e* was selected to be 0.050 radians, and *c* was selected to be 0.80.

C6.8.2.5 Rehabilitation Measures

When strengthening or stiffening a shear wall, the designer is reminded to evaluate the strength and stiffness of floor diaphragms and their connections to the shear wall. Also, the strength and stiffness of the foundation supporting the shear wall must be evaluated. All connections between new and existing structures should satisfy the requirements in Section 6.4.6 of the *Guidelines*.

The addition of wall boundary elements to increase the flexural strength of a shear wall requires a careful evaluation of the ratio between the wall's shear strength and the increased shear forces required to develop the flexural strength of the wall. In several cases the wall shear strength will need to be increased to ensure that the shear wall will exhibit ductile flexural behavior if it is overloaded.

Confinement jackets may be added to shear wall boundaries to either increase the deformation capacity of the wall, or increase both the wall flexural strength and deformation capacity. In the latter case, the shear capacity of the wall must be checked as noted above. Research results have shown that effective confinement of wall boundaries can be achieved by the use of concrete jackets, steel jackets, or fiber wraps (Iglesias, 1987; Aguilar et al., 1989; Jirsa et al., 1989; Aboutaha et al., 1994; Katsumata et al., 1988; Priestley et al., 1992).

For shear walls that have a shear capacity less than the shear required to develop the flexural capacity of the wall, a designer may elect to reduce the flexural capacity of the wall. A decision to reduce the lateral load capacity of a structure should be carefully evaluated to be sure that the improved ductile behavior of the structure more than compensates for its reduced strength.

In shear critical walls where the designer does not want to reduce the flexural strength of the wall, the shear capacity of the wall can be enhanced by increasing the thickness of the web of the wall. The extra web thickness should be reinforced with horizontal and vertical steel. Before casting the new concrete, the surface of the existing wall should be roughened and dowels should be placed to ensure that the old and new concrete will work together. In lieu of increasing the wall thickness, recent research (Ehsani and Saadatmanesh, 1994) has shown that the addition of carbon fiber bands is effective in increasing the shear strength and stiffness of existing walls.

As discussed in Section 6.5 of the *Guidelines*, steel or reinforced confinement jackets can be used to increase the shear capacity and confinement in beams and columns. These same procedures are effective for improving the inelastic behavior of coupling beams and RC columns supporting discontinuous shear walls. Even though these members may not initially appear to be shear critical, their shear strength may decrease under reversed cyclic loading. The use of a confinement jacket will either prevent or at least significantly delay the decrease in the member's shear strength with cycling.

Even the addition of confinement jackets may not be sufficient to improve the response of an RC column supporting a discontinuous shear wall to a satisfactory level. In such cases, it may be necessary to significantly change the demands placed on those columns by changing the layout of the structure. Shear walls could be added at other locations in the structure, but a more effective means will be to add new elements below the discontinuous wall. One procedure is to add a concrete infill between the existing columns (Kahn and Hanson, 1979; Jirsa et al., 1989; Valluvan et al., 1994). A second procedure is to add steel bracing members between the columns (Bush et al., 1991; Goel and Lee, 1990). For both cases, the new members will need to be evaluated by the procedures given in the *Guidelines* for new construction.

C6.9 Precast Concrete Shear Walls

C6.9.1 Types of Precast Shear Walls

In the past, precast wall systems have seldom been used as primary lateral-load-resisting elements for structures located in high seismic risk zones of the United States. There has been a general belief that precast construction was inherently less ductile than monolithic construction, and thus should not be used for structures that may experience moderate or severe earthquake excitation during their service life.

In more modern seismic building codes, precast shear wall construction is permitted in high seismic risk zones if it can be shown by experiment or analysis that the lateral-load-resisting characteristics of the precast system are at least equal to those of a similar cast-in-place shear wall system. This design requirement has led to a type of precast shear wall construction known as cast-in-place emulation. For this design approach, the connections between the precast components are detailed such that inelastic action will occur away from the connections. Since the precast components can be reinforced and detailed similarly to monolithic walls, then the inelastic response of the precast system should be identical to that of a cast-in-place system. Although this emulation design approach may be effective and predictable, this approach has a tendency to undermine the cost-effectiveness of precast concrete systems.

As a result of the recent National Science Foundation-sponsored research program entitled PRESSS (PREcast Seismic Structural Systems) (Priestley, 1995), there is now some experimental and analytical evidence to indicate that precast structures that do not emulate monolithic cast-in-place construction may be used to resist severe earthquake loading. In this new design philosophy, known as "jointed construction," some of the joints between precast members are designed to deform inelastically under large lateral loads, thereby providing ductility and energy dissipation to the structural system. These ductile joints between precast elements may consist of both vertical and horizontal connections between panels.

Precast shear walls in several older structures cannot be classified as cast-in-place emulation because the joints were not designed to force all inelastic action away from the connection region. Also, these older precast walls would not satisfy the more modern definition of jointed construction because the connections were not designed with special elements intended to absorb energy in a stable ductile manner. This older type of jointed construction was not permitted in high seismic zones, and the designer will need to be careful when assessing its deformation capacity. For these older precast shear walls, continuity splices between the horizontal and vertical web reinforcement of the wall panels was normally obtained by a simple interconnection of the bars protruding from adjacent wall segments. Because the cast-in-place connections between panels are too short to satisfy the requirements for a tension lap splice, the bars may have been either hooked around each other to create a mechanical interlock, or fillet welded along their short lap length. The larger vertical bars commonly used along the vertical edges of a wall panel would have required special splicing hardware. A variety of proprietary rebar splice connectors have been used in older construction and may still be used in modern precast wall construction.

Tilt-up walls are considered to be a special case of jointed construction. The in-plane shear strength of these walls should be evaluated as a force-controlled action. Failure of the connection between the tilt-up wall and the roof diaphragm has been the most common type of failure observed for these types of structures during significant seismic loading. If that connection fails, the wall panel is subjected to out-of-plane forces and deformations that could cause it to collapse. Thus, the designer is cautioned to carefully check the connection between the wall and the roof diaphragm.

C6.9.2 Precast Concrete Shear Walls and Wall Segments

C6.9.2.1 General Modeling Considerations

The general analytical modeling considerations for precast concrete shear walls are very similar to those for monolithic cast-in-place shear walls. Therefore, the reader is referred to Section C6.8.2.1.

In addition to modeling the precast wall panels, the designer will need to include an analytical model to represent deformations in connections between the precast panels. Such connection models can only be avoided if the connections are designed and detailed to remain elastic, and all inelastic response of the precast wall system will take place in the precast panels.

C6.9.2.2 Stiffness for Analysis

The *Guidelines* offer two alternatives for including the stiffness of the connections between precast panels in the analytical model. One option would be to modify the analytical model used for the wall panels to represent the stiffness of the assembled wall panels and connections. The second option is to keep the same stiffness parameters as used for monolithic walls, but add a separate element to represent the stiffness of the connection.

A. Linear Static and Dynamic Procedures

No commentary is provided for this section.

B. Nonlinear Static and Dynamic Procedures

A general discussion of nonlinear procedures for shear walls and wall segments is given in Section C6.8.2.2B. Most of that discussion for monolithic concrete shear walls and wall segments is also applicable to precast walls and wall segments.

When using the basic load-deformation curves given in Figure 6-1, the deformation values (*x*-axis) are to be defined as either plastic hinge rotation or drifts, depending on whether the wall's (or wall segment's) inelastic response is governed by flexure or shear. Plastic hinge rotations are used where flexure governs the inelastic response for shear walls and wall segments. A sketch of the first story of a deformed shear wall governed by flexure is given in Figure 6-2. The length of a plastic hinging region in an RC member is typically defined to be somewhere between 0.5 and 1.0 times the effective flexural depth of the member. For RC members where shear deformations are significant, the plastic hinging length tends toward the upper end of this range, and vice versa. Therefore, for the shear walls the plastic hinging region will extend very close to, if not beyond, one story height of the member. In these cases it is appropriate to limit the length of the plastic hinging zone to one story height. For wall segments, which often have small length-to-depth ratios, the plastic hinging zone may extend to mid-length of the member. For those cases, the length of the plastic hinging zone is limited to one-half the length of the member.

For members whose inelastic response is controlled by shear, it is more appropriate to use drifts as the deformation value in Figure 6-1(b). For shear walls, this drift is actually the story drift as shown in Figures 6-3. For wall segments, the member drift is used.

For monolithic construction, values for the hinge rotation values *a* and *b*, described in Figure 6-1(a), are given in Table 6-17, and the drift values *d* and *e*, described in Figure 6-1(b), are given in Table 6-18. For cast-in-place emulation types of precast wall construction, the full tabulated values are used. For jointed construction, the tabulated values are to be reduced by 50%. This is a severe reduction, but the design engineer can use a smaller reduction if there is experimental evidence to support the use of higher values.

C6.9.2.3 Design Strengths

The discussion of the calculation of yield and nominal strengths given in Section C6.8.2.3 is applicable to precast shear walls and wall segments that are classified as cast-in-place emulation. For all types of jointed construction, the strength of the precast shear wall will be significantly affected by the strength of the connections. Thus, the connection strength must be evaluated as described in the *Guidelines*. Special consideration must be given to the type of splicing used for the reinforcement present in the connection. In many cases the strength of the splice will govern the strength of the connection.

C6.9.2.4 Acceptance Criteria

A. Linear Static and Dynamic Procedures

As previously stated, precast shear walls that emulate cast-in-place construction and wall elements with precast panels shall be evaluated by the same procedure as used for cast-in-place shear walls and wall elements. For jointed construction, the *m* values, which give a measure of a member's ductility, shall be reduced to 50% of the values given in Tables 6-19 and 6-20. This severe reduction in the available ductility can be changed if the designer has access to experimental evidence that justifies a higher *m* value.

B. Nonlinear Static and Dynamic Procedures

Inelastic response is only acceptable for those actions listed in Tables 6-17 and 6-18. A detailed discussion of the deformation values given in these tables was presented in Section C6.8.2.4B. For jointed construction, the deformation values are reduced to 50% of the tabulated values because of uncertainty about the inelastic behavior of older versions of this type of construction.

C6.9.2.5 Rehabilitation Measures

As the *Guidelines* note, precast concrete shear walls may suffer from some of the same problems experienced by monolithic shear walls. Therefore, most of the rehabilitation measures described in Section 6.8.2.5 are applicable to precast shear walls.

Connections between precast panels and between the panels and the foundation offer an additional set of problems in precast walls. Most of the deficiencies in strength at the connections can be rehabilitated through the use of supplemental mechanical connectors or cast-in-place connections doweled into the adjacent members. Rather than add ductile supplemental connections, the designer should attempt to make the connections stronger than the adjacent panels, and thus force any inelastic behavior into those panels. The designer is cautioned to consider out-of-plane forces and deformations when designing and detailing supplemental panel-to-panel connections and panel-to-foundation or panel-to-floor diaphragm connections.

C6.10 Concrete Braced Frames

C6.10.1 Types of Concrete Braced Frames

Reinforced concrete braced frames are relatively uncommon in existing construction, and are seldom recommended for use as ductile earthquake resisting systems. They are sometimes used in the United States for wind-bracing systems, where inelastic response is not anticipated. Examples of concrete braced frames have been identified in other countries. These bracing systems may have provided necessary stiffness and strength that saved many concrete frames during the 1985 Mexico City earthquake, but there are also many examples of poor performance in the same systems during this earthquake. In general, these types of elements are not recommended for regions of moderate and high seismic activity.

C6.10.2 General Considerations in Analysis and Modeling

Braced frames resist lateral forces primarily through tension and compression in the beams, columns, and diagonal braces. Therefore, it is usually acceptable to model these frames as simple trusses. As with other reinforced concrete framing systems, the analysis model must recognize the possibility for failure along the length of the component (as in tension failure of reinforcement splices) or in connections.

C6.10.3 Stiffness for Analysis

C6.10.3.1 Linear Static and Dynamic Procedures

If the braced frame is modeled as a truss, it is acceptable for beams, columns, and braces to use the recommended axial stiffnesses for columns from Table 6-4. Joints may be modeled as being rigid.

C6.10.3.2 Nonlinear Static Procedure

The writers were unable to identify test data related to reinforced concrete braced frames. However, the braced-frame action of this element is expected to be similar in many regards to that for infilled frames modeled using the braced-frame analogy. Therefore, it is acceptable to use the general modeling parameters from Section 6.7.

C6.10.3.3 Nonlinear Dynamic Procedure

The writers were unable to identify test data related to reinforced concrete braced frames. The analyst must use engineering judgment in establishing the analysis model for the NDP.

C6.10.4 Design Strengths

The general procedures of ACI 318 for calculation of compressive and tensile strength are applicable, subject to the guidelines of Section 6.4.2.

C6.10.5 Acceptance Criteria

Existing construction of concrete braced frames is unlikely to contain details necessary for ductile response. These details include: (1) in compression members, adequately detailed transverse reinforcement to confine concrete and restrain longitudinal reinforcement from buckling; (2) in tension members, reinforcement splices having strength sufficient to develop post-yield tension behavior in longitudinal reinforcement; and (3) in connections, adequate anchorage for longitudinal reinforcement. Where these details are not provided, actions should be defined as being force-controlled.

C6.10.6 Rehabilitation Measures

Rehabilitation measures that are likely to improve response of existing concrete braced frames include the following:

- Jacketing of existing components, using steel, reinforced concrete, or composites to improve continuity and ductility
- Various measures to improve performance of lap splices, including chipping cover concrete and welding
- Removal of the diagonal bracing, leaving a moment-resisting frame, which must then be checked according to procedures in Section 6.5
- Addition of steel braces, walls, buttresses, or other stiff elements to control lateral drift and protect the existing braced frame
- Infilling of the braced frame with reinforced concrete, either with the brace in place, or after removal of the brace
- Modification of the structural system through such techniques as seismic isolation

C6.11 Concrete Diaphragms

Cast-in-place diaphragms have had a relatively good performance record in worldwide earthquakes when the configuration was not irregular and when the length-to-width ratio was relatively small (less than three to one). Thin concrete slabs associated with one-way beam and joist systems are limited in diaphragm shear capacity and become more suspect as the length-to-width ratio increases.

C6.11.1 Components of Concrete Diaphragms

No commentary is provided for this section.

C6.11.2 Analysis, Modeling, and Acceptance Criteria

No commentary is provided for this section.

C6.11.3 Rehabilitation Measures

No commentary is provided for this section.

C6.12 Precast Concrete Diaphragms

C6.12.1 Components of Precast Concrete Diaphragms

Precast concrete diaphragms contain a variety of different components that have been used at different times and in different geographic regions. The precast industry first began to produce components in the early 1950s. Many of the first components were reinforced with mild steel and utilized concrete strengths in the range of 3000 psi. Rectangular beam, inverted tee beam, L beam, column, channel shape, slab, double tee, and single tee components (reinforced, prestressed, and post-tensioned) were available in most regions of the United States by 1960. The connections utilized are generally brittle, with varying amounts of limited ductility. Concrete strengths were then routinely specified at 6000 psi or more to facilitate quick turnaround of casting facilities. Only a small percentage of these systems were designed with ultimate level seismic forces in mind. Diaphragms rarely had a composite topping slab poured on them if they were at the roof level, but most floor systems do have poured composite topping slabs.

Topped diaphragms may have the following seismic deficiencies:

- Inadequate topping thickness and general reinforcement
- Brittle connections between components
- Excessive diaphragm length-to-width rations
- Little or no chord/connector steel
- Inadequate shear transfer capacity at boundaries
- Inadequate connections and bearing length of components at supports
- Corrosion of connections

Whether or not the diaphragms were initially designed for seismic forces, the performance of precast diaphragms during the 1994 Northridge earthquake demonstrated that the following items should be reviewed as part of an evaluation/rehabilitation program.

- **Diaphragm Rigidity**. Diaphragms experience relatively large displacements due to the yielding of reinforcing used as temperature steel in the deck, the yielding of collectors and chords, and, in some cases, the long length-to-depth ratios. Brittle failure of individual component-to-component connections will also contribute to greater-than-expected displacements. Diaphragm displacements may be much larger than associated shear wall drifts; therefore, the distribution of seismic forces will be much different than that determined from a rigid diaphragm assumption. Columns assumed to be non-seismic-resisting have failed because of the displacements that they experienced.

- **Complete Load Paths.** The joints or seams between spanning members and the joints along the ends of such members are generally covered with thin concrete overlays and are often lightly reinforced. The structural response of the diaphragm may be strongly influenced by the action along these seams. Critical sections may require reinforcement.

- **Collector Design.** The chord forces and diaphragm collector forces should be designed to have limited yielding, or designed with confinement steel similar to ductile axial column members. Initial tension yielding causes a situation where subsequent cyclic compression forces may buckle the reinforcement. This type of failure was observed following the 1994 Northridge, California earthquake (Corley, 1996). Additionally, it was observed that shear wall/collector connections failed. These failures could be the primary collapse mechanism, or could be secondary to other factors. It is clear that collector-to-shear-wall connections are critical; they should be designed for ductility where possible, with strength commensurate with the ductility assumed. The effects of cyclic tension/compression actions should be recognized in the design of confinement steel. Also, it is important to recognize the effects of shear wall rocking and rotation on the collector connection. This action, along with the fracture potential of bulking bars, has not generally been recognized.

- **Vertical Acceleration.** Gravity-loaded long-span precast members may be vulnerable to vertical accelerations at sites close to fault systems. Corley states that “A combination of gravity load and vertical acceleration may have caused failure of some inverted tees." Other observers have noted this possibility with respect to different members.

C6.12.2 Analysis, Modeling, and Acceptance Criteria

No commentary is provided for this section.

C6.12.3 Rehabilitation Measures

Rehabilitation measures for precast concrete diaphragms are difficult and, in many cases, expensive. The installation of new shear walls or rigid braces can be very effective, in that demands on components, elements, and connections can be greatly reduced. Experience with other techniques is limited. In the case of untopped roof diaphragms, removal of the precast concrete deck should be considered. The installation of a modern seismic-resisting system may be economical in some cases.

C6.13 Concrete Foundation Elements

C6.13.1 Types of Concrete Foundations

This section provides guidelines primarily for seismic analysis, evaluation, and enhancement of concrete foundation elements that occur in buildings with structural frames, or concrete or masonry shear and bearing walls. Selected portions of these guidelines may also be applicable to other structural systems and to foundation elements of other structural materials (e.g., timber or steel piles).

C6.13.2 Analysis of Existing Foundations

The simplifying assumptions regarding the base conditions for the analytical model are similar to those required for gravity load analyses. The procedures described for more rigorous analyses are considered to provide more rational representation of the soil-structure and soil-pile interaction under lateral loading. These more rigorous procedures are therefore recommended to provide a higher confidence level for the more demanding Performance Levels. Since the net effect of these procedures is generally to reduce stresses in the building, but to increase displacements, these procedures may make it possible to accept an otherwise deficient stiff building if the resulting displacements are within allowable limits.

C6.13.3 Evaluation of Existing Condition

In the absence of dependable construction drawings, confirmation of the size and detailing of existing foundations may not be possible without resorting to invasive procedures. For larger or important buildings, limited demolition of selected foundations may be necessary where adequate construction documentation is not available. Drawings are more likely to be available for buildings with deep foundations. For most buildings with shallow foundations, if drawings are not available, selected exposure of representative footings may be required to establish size and depth. Conservative assumptions regarding reinforcement may be made, considering code requirements and local practice at the time of design. In case of doubt, it can be assumed that the foundation elements were designed adequately to resist the actual gravity loads to which the building has been subjected, although the actual factor of safety will still be in doubt.

Because of the difficulty associated with the exposure and repair of potential seismic damage to foundations, current preferred practice is to preclude damage by ensuring the yielding occurs in the columns or walls above the foundation. For this reason, it is stipulated that the existing foundation elements be evaluated with the smaller of the unreduced design forces or the forces based upon the capacity of the supported columns or walls.

C6.13.4 Rehabilitation Measures

The seismic rehabilitation or enhancement of foundation elements in existing buildings is generally an expensive and disruptive process. Limited accessibility, and the difficulty and risks associated with strengthening existing foundation elements that are supporting the building gravity loads, often lead the engineer to search for a more cost-effective solution. In many cases, when analysis indicates that existing foundation elements may be subjected to excessive seismic force, the deficiency may be reduced or mitigated by new vertical lateral-force-resisting elements (e.g., bracing or shear walls) that will divert the seismic forces to new foundation elements or to other lightly loaded existing elements. While the strengthening techniques described in this chapter are considered to be practical and feasible, the designer is encouraged to develop and evaluate alternative mitigation measures that may be more cost-effective for the building owner. Accepting performance that allows for permanent soil deformation below the footings will reduce the rehabilitation cost.

C6.13.4.1 Rehabilitation Measures for Shallow Foundations

Spread footings generally include individual column footings and continuous strip footings supporting wall loads. Existing small or lightly loaded column footings may be unreinforced; larger and heavier loading footings will have a horizontal curtain of steel near the bottom of the footing. Strip footings are generally composed of square or rectangular continuous footings designed so as to not exceed the allowable soil bearing pressures. A concrete stem wall may extend above the footing to support the wall above and may have a ledge to support the floor slab. The footing and the stem wall may be reinforced, or may have a few continuous horizontal bars at the bottom of the footing and one or two horizontal bars at the top of the stem wall. More recent or better-designed existing wall footings will have vertical reinforcement in one or both vertical faces of the stem wall.

A reinforced concrete shear wall or a concrete frame with an infilled concrete or masonry wall may have a combination footing, consisting of a strip footing under the wall and a monolithic spread footing at each end under the columns or boundary members of the shear wall.

Concrete mats are large footings that support a number of columns and walls and rely on the flexural stiffness of the mat to distribute the supported loads to the soil, or the piles or piers. Mats will usually have a horizontal curtain of reinforcement at the bottom and an additional curtain at the top of the mat; they may or may not have any distributed vertical reinforcement.

If the design seismic forces in a footing result in load combinations that exceed the deformation limits or the allowable soil pressure, the existing footing must be enlarged, or additional lateral-load-resisting elements may be added, to reduce the soil bearing pressure under the footing to allowable levels.

An existing column footing may be enlarged by a lateral addition if proper care is taken to resist the resulting shears and moments. The original footing will continue to support the load at the time of extension, and the extended footing will participate in the support of the subsequent loads. If the existing footing is founded on poor soil but more competent bearing strata occur at reasonable depth, it may be feasible to convert the spread footing into a pier-supported footing by drilling through the footing and providing cast-in-place reinforced concrete piers under the footing. If the existing footing has inadequate shear or moment capacity for the resulting forces from the new piers, the capacity may be enhanced by new concrete to increase the depth of the footing.

If the seismic rehabilitation criteria result in overturning moments that cause uplift in an existing spread footing, tension hold-downs can be provided. Because of their slenderness, the hold-downs may be assumed to resist tension only. Reversed movements from these tension ties may require the addition of horizontal reinforcement in new concrete fill at the top of the footing. The design engineer must consider whether uplift or rocking will cause unacceptable damage in the building.

A typical perimeter wall footing may also be strengthened by procedures similar to those described above for individual column footings. An alternative strengthening procedure commonly utilized for continuous footings is underpinning. Underpinning is generally accomplished by progressive incremental excavation under an existing footing, and replacement of the excavated material with new concrete to provide a larger footing. The lateral extension and the depth of the underpinning are generally selected so that the concrete may be assumed to be in compression and reinforcement of the underpinning is not required. Underpinning may also be used to provide tension hold-downs for an existing wall footing subject to uplift forces from seismic overturning moments. A pair of drilled and grouted tension ties is provided at each end of the wall footing and anchored into a new cap that is constructed by underpinning the end of the wall. If significant tensile forces are to be resisted, it may be necessary to provide concrete wing walls on either side of the wall, extending vertically from the new cap to a length adequate to transfer the tensile uplift force from the existing wall by dowels and shear friction.

Concrete mats are typically analyzed as isotropic plates with concentrated loads on an elastic foundation, and are sensitive to the assumed subgrade modulus for the soil. Because of the difficulty and cost associated with strengthening an existing mat foundation, it is recommended that, if any of the above deficiencies are identified, the assumed soil properties be reviewed and additional geotechnical investigations be made to determine if more favorable properties can be justified. Similarly, if the analysis indicates localized soil bearing pressures that exceed allowable values, the geotechnical consultants should be asked to review the allowable values with the actual conditions of loading and lateral confinement. The engineer and owner should also consider whether permanent soil deformation is acceptable.

If it is feasible to increase the depth of the mat with a reinforced concrete overlay, both the flexural reinforcement and vertical shear capacities may be enhanced. This may be the only retrofit procedure feasible for a deficient mat. A practical alternative to retrofitting would be to evaluate the consequences of allowing limited yielding of the reinforcement and/or cracking of the concrete under the design seismic loading. This evaluation can be performed with available nonlinear analysis computer codes, or can be approximated with linear elastic analyses by progressively “softening” the yielding elements.

If the soils under the mat are found to be compressible or otherwise unsuitable, pilings driven through drilled holes in the mat foundation to competent soil strata can be used. This is sometimes employed in new construction to offset an abrupt variation in the soil profile under the mat. In existing buildings, care must be exercised in design and construction so as not to damage the existing mat reinforcement, and deformation compatibility must be maintained under the design loadings without overstressing the mat.

C6.13.4.2 Rehabilitation Measures for Deep Foundations

Concrete piles or piers are generally surmounted by a concrete cap that supports the base of a column or wall. A concrete pedestal is sometimes utilized to raise the base of the column to a more convenient elevation and/or to achieve a better distribution of loads to the pile or pier cap.

Concrete piles may be precast, or precast and prestressed, and are driven with or without predrilling of the soil. The piles are considered to be point bearing if they are driven to “refusal” in rock or other hard material, and as friction piles if the loads are transferred to soil by cohesion or friction.

Concrete piers are generally designed as reinforced concrete columns, and constructed by placing the reinforcement and concrete in either open or cased drilled holes. Proprietary systems are in use that utilize thin metal shells driven with a steel mandrel in lieu of drilling.

Anchorage of the piles or piers into the cap may vary from simple embedment of several inches without dowels to complete development of the vertical reinforcement into the cap. Pile and pier caps are designed to resist the moments and shears from the pile or pier reactions. Typically, the caps are designed with sufficient depth to resist the shear without reinforcement, and a curtain of horizontal reinforcement near the bottom of the cap is designed to resist the flexural moments. For severe pile loads, or when the depth of the cap is limited, vertical shear reinforcement may be required, and a horizontal curtain of reinforcement may be provided near the top of the cap to anchor the shear reinforcement.

If the existing piles or piers are found to be deficient in vertical load capacity, the capacity can be increased by adding additional piles or piers. If the new elements are added with an extension of the existing cap, the existing cap may have to be strengthened to resist the moment and shear from the additional piles or piers. The new piles or piers will only participate in the resistance of vertical loads subsequent to their construction. In some cases, where the existing foundation is judged to be seriously deficient, it may be cost-effective to provide temporary shoring to permit removal and complete replacement of the foundation.

A common problem in the seismic rehabilitation of existing buildings is uplift on the existing foundation. If the existing piles or piers and/or their anchorages to the caps are inadequate for the design uplift forces, new elements can be provided to resist the tensile uplift forces. If new piles or piers are required to resist the vertical compressive forces, it may be feasible to design these new elements and to strengthen the cap to resist the uplift forces. If new elements are not required for the compressive forces, it may be possible to provide the necessary uplift capacity by means of hold-downs drilled through the existing caps. The hold-downs consist of high-tensile-strength steel rods or strands, anchored by grouting in firm material at the bottom and in the concrete cap at the top. The existing caps need to be investigated and strengthened, if necessary, for the reverse flexural moment resulting from the uplift forces.

Inadequate moment capacity of the existing cap reinforcement can be improved by adding additional concrete to increase the depth of the existing cap. This has the effect of increasing the effective depth of the cap, and thus reducing the tensile stress in the existing reinforcement. The top of the existing cap should be roughened and provided with shear keys or dowels to resist the horizontal shear at the interface. The additional depth that can be provided may be limited to functional restrictions (e.g., interference with the floor slab), or the additional weight that can be supported by the existing piles or piers. It should be noted that increasing the depth of the cap may decrease the effective length of the column above, and require a revision in the relative rigidity calculations for distribution of lateral loads. Additionally, it should be noted that this procedure may not be applicable to caps supporting columns that are assumed to be pinned at their base, since the additional cap depth may result in undesirable fixity of the column base.

Where the moments in the existing columns are large enough to cause uplift in the piles or piers, reversed moments will occur in the cap, requiring tensile reinforcement near the top surface. If this reinforcement is absent or deficient, the required reinforcement can be provided in a new concrete overlay to the existing cap. To improve the effectiveness of the new reinforcement, it may be necessary to drill and grout some of the bars through the existing column. If this is not feasible, the effective transfer of tensile forces to the new reinforcement must be investigated by the strut and tie method, or other rational procedures. Alternatively, temporary shoring of the column loads can be provided so that the existing column reinforcement can be exposed and the new horizontal reinforcement placed effectively. As discussed in the previous paragraph, if the additional depth of cap significantly reduces the effective length of the column, the distribution of the lateral load shears may have to be reevaluated.

Inadequate vertical shear capacity in the existing caps can also be improved by providing additional depth to the caps. Since it is not considered feasible to provide new vertical shear reinforcement in an existing cap, if the necessary capacity cannot be obtained by increasing the depth of the cap, the only available alternatives may be to remove and replace the existing cap with an appropriate new cap, or to provide new lateral-load-resisting elements (e.g., shear walls or braced frames) that will reduce the forces to be resisted by the existing cap to allowable levels.

If the vertical reinforcement in the existing piles or piers is adequately developed into the caps, then the pile or pier will provide lateral force resistance by flexural bending. The lateral load capacity of these elements can be approximated by assuming fixity at a depth below the cap equal to about ten diameters for very soft soils and five diameters for very firm soils. The moment or shear capacity can then be calculated assuming full or partial fixity at the cap. The pile or pier capacity is compared with the portion of the design lateral load to be resisted by the piles or piers, as determined by consideration of deformation compatibility with the portion resisted by passive pressure of the soil on the cap. If the total effective capacity of the piles or piers and the cap is inadequate, the practical alternatives are to enhance the passive pressure capacity of the cap; to remove and replace the existing cap with or without the addition of new piles or piers; or to reduce the lateral forces on the existing foundation elements by providing additional resisting elements.

Pile and pier foundations resist lateral forces by means of passive soil pressure on the caps or by bending of the piles or piers. If the anchorage of the existing piles or piers to the caps is inadequate or questionable in regard to development of moments in the piles or piers, passive soil pressure on the caps may constitute the principal lateral load resistance of the foundation. The total resisting capacity of the foundation system will include passive pressure on tie beams and perimeter walls extending below grade. In order to mobilize the total resisting capacity of the existing foundation system, it is important that all of the resisting elements be properly interconnected. This connection may be accomplished by a competent slab at or near the top of the caps, or by adequate tie beams to affect the distribution. If the existing total capacity is inadequate, the alternatives include enhancing the passive resistance of the soil; increasing the contact areas of the caps, tie beams, and perimeter walls; or a combination of these alternatives.

The passive resistance of the soil can be enhanced by a number of techniques, such as compaction and/or intrusion grouting with appropriate chemicals or soil/cement mixtures, as described in Chapter 4.

C6.14 Definitions

No commentary is provided for this section.

C6.15 Symbols

No commentary is provided for this section.

C6.16 References

Aboutaha, R. S., Engelhardt, M. D., Jirsa, J. O., and Kreger, M. E., 1993, “Seismic Strengthening of R/C Columns Using Steel Jackets,” American Society of Civil Engineers Structures Congress, Irvine, California.

Aboutaha, R. S., Engelhardt, M. D., Jirsa, J. O., and Kreger, M.E., 1994, “Seismic Retrofit of R/C Columns with Inadequate Lap Splices,” American Society of Civil Engineers Structures Congress, Atlanta, Georgia.

Aboutaha, R., et al., 1994, “Seismic Shear Strengthening of R/C Columns Using Rectangular Steel Jackets,” *Proceedings*, Fifth U.S. National Conference on Earthquake Engineering, Earthquake Engineering Research Institute, Oakland, California, Vol. III, pp. 799–808.

Abrams, D. P., and Angel, R., 1993, “Strength, Behavior, and Repair of Masonry Infills,” *Proceedings*, American Society of Civil Engineers Structures Congress, Irvine, California.

ACI, 1988, “Recommendations for Design of Slab-Column Connections in Monolithic Reinforced Concrete Structures,” ACI 352-88, ACI-ASCE Committee 352, *ACI Structural Journal*, American Concrete Institute, Detroit, Michigan, Vol. 85, No. 6, pp. 675–696.

ACI, 1989, *Building Code Requirements for Reinforced Concrete*, American Concrete Institute Report No. 318-89, Detroit, Michigan.

ACI, 1989, *In-Place Methods for Determination of Strength of Concrete*, American Concrete Institute Report ACI 228-89, Detroit, Michigan.

ACI, 1991, *Strength Evaluation of Existing Concrete Buildings*, Report ACI 437-91, American Concrete Institute, Detroit, Michigan.

ACI, 1994, *Guide for Evaluation of Concrete Structures Prior to Rehabilitation*, Report ACI 364-94, American Concrete Institute, Detroit, Michigan.

ACI, 1995a, *In-Place Methods to Estimate Concrete Strength*, Report ACI 228.1R-95, American Concrete Institute, Detroit, Michigan.

ACI, 1995b, *Building Code Requirements for Structural Concrete (ACI 318-95) and Commentary (ACI 318R-95)*, American Concrete Institute, Detroit, Michigan.

ACI, various years, Standards with the following numbers: 364.1R-94, 201.2R-92, 355.1R-91, 437-94, American Concrete Institute, Detroit, Michigan.

Aguilar, J., et al., 1989, “The Mexico Earthquake of 1985 - Statistics of Damage and of Retrofitting Techniques in Reinforced Concrete Buildings,” *Earthquake Spectra*, Earthquake Engineering Research Institute, Oakland, California, Vol. 5, No. 1, pp. 145–151.

Alama, M. S., and Wight, J. K., 1992, *Numerical Modeling and Parametric Investigation of R/C Coupled Wall-Frame Systems under Seismic Loading*, Report No. UMCEE 92-20, Department of Civil and Environmental Engineering, University of Michigan, Ann Arbor, Michigan, p. 251.

Alcocer, S. M., and Jirsa, J. O., 1993, “Strength of Reinforced Concrete Frame Connections Rehabilitated by Jacketing,” *ACI Structural Journal*, American Concrete Institute, Detroit, Michigan, Vol. 90, No. 3, pp. 249–261.

Ali, A. and Wight, J. K., 1991, “RC Structural Walls with Staggered Door Openings,” *Journal of the Structural Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 117, No. 5, pp. 1514–1531.

Altin, S., Ersoy, U., and Tankut, T., 1992, “Hysteretic Response of Reinforced Concrete Infilled Frames,” *Journal of the Structural Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 118, No. 8, pp. 2133–2150.

Aoyama, H., Kato, D., Katsumara, H., and Hosokawa, Y., 1984, “Strength and Behavior of Postcast Shear Walls for Strengthening of Existing Reinforced Concrete Buildings,” *Proceedings*, Eighth World Conference on Earthquake Engineering, San Francisco, California, Earthquake Engineering Research Institute, Oakland, California.

ASCE, 1990, *Standard Guideline for Structural Condition Assessment of Existing Buildings*, Standard 11-90, American Society of Civil Engineers, New York, New York.

Aschheim, M., and Moehle, J. P., 1992, S*hear Strength and Deformability of RC Bridge Columns Subjected to Inelastic Cyclic Displacements*, Report No. UCB/EERC-92/04, Earthquake Engineering Research Center, University of California at Berkeley, California.

ASTM, 1990, *Standard Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete*, ASTM C 42, American Society for Testing and Materials, Philadelphia, Pennsylvania.

ASTM, 1995, *Standard Practice for Examination and Sampling of Hardened Concrete in Constructions*, ASTM C 823, American Society for Testing and Materials, Philadelphia, Pennsylvania.

ASTM, latest edition, Standards with the following numbers: A370, A470, A751, C597, C803, C805, C856, C876, American Society of Testing Materials, Philadelphia, Pensylvania.

ATC, 1988, *Rapid Visual Screening of Buildings for Potential Seismic Hazards*, prepared by Applied Technology Council, (Report ATC-21), for the Federal Emergency Management Agency (FEMA Report 154), Washington, D.C.

AWS, 1992, *Structural Welding Code - Reinforcing Steel*, ANSI/AWS Standard D1.4-92, American Welding Society, Miami, Florida.

Aycardi, L. E., Mander, J. B., and Reinhorn, A. M., 1994, "Seismic Resistance of Reinforced Concrete Frame Structures Designed Only for Gravity Load: Part II - Experimental Performance of Subassemblages," *ACI Structural Journal*, American Concrete Institute, Detroit, Michigan, Vol. 91, No. 5, pp. 552–563.

Bachman, H., and Linde, P., 1995, "Dynamic Ductility Demand and Capacity Design of Earthquake-Resistant Reinforced Concrete Walls," *Recent Developments in Lateral Force Transfer in Buildings*, ACI SP-157, American Concrete Institute, Detroit, Michigan, pp. 117–141.

Bartlett, F. M., and Sexsmith, R. G., 1991, "Bayesian Technique for Evaluation of Material Strengths in Existing Bridges," *ACI Materials Journal*, American Concrete Institute, Detroit, Michigan.

Bartlett, F. M., and MacGregor, J. G., 1995, "Equivalent Specified Concrete Strength from Core Test Data," *ACI Concrete International*, American Concrete Institute, Detroit, Michigan.

Bass, R. A., Carrasquillo, R. L., and Jirsa, J. O., 1989, "Shear Transfer Across New and Existing Concrete Interfaces," *ACI Structural Journal*, American Concrete Institute, Detroit, Michigan, Vol. 86, No. 4, pp. 383–393.

Beres, A., White, R. N., and Gergely, P., 1992, *Seismic Behavior of Reinforced Concrete Frame Structures with Nonductile Details: Part I - Summary of Experimental Findings of Full Scale Beam-Column Joint Tests*, Report NCEER-92-0024, National Center for Earthquake Engineering Research, State University of New York at Buffalo, New York.

Beres, A., El-Borgi, S., White, R. N., and Gergely, P., 1992, *Experimental Results of Repaired and Retrofitted Beam-Column Joint Tests in Lightly Reinforced Concrete Frame Buildings*, Report NCEER-92-0025, National Center for Earthquake Engineering Research, State University of New York at Buffalo, New York.

Bertero, V. V., 1987, "Observations of Structural Pounding," *The Mexico Earthquakes of 1985, Factors Involved and Lessons Learned*, edited by Cassaro, M.A., and Martinez Romero, E., American Society of Civil Engineers, New York, New York, pp. 264–278.

Bertero, V. V., et al., 1995, *Seismological and Engineering Aspects of the 1995 Hyogoken-Nanbu (Kobe) Earthquake*, Report No. UCB/EERC-95/10, Earthquake Engineering Research Center, University of California, Berkeley, California.

Bett, B. J., Klingner, R. E., and Jirsa, J. O., 1988, "Lateral Load Response of Strengthened and Repaired Reinforced Concrete Columns," *ACI Structural Journal*, American Concrete Institute, Detroit, Michigan.

Brookes, C. L., Buchner, S. H., and Mehrkar-Asl, S., 1990, "Assessment of Stresses in Post-Tensioned Concrete Bridges," *Bridge Management - Inspection, Maintenance, Assessment, and Repair*, Elsevier Applied Science, London, United Kingdom.

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.

Bush, T. D., Talton, C. R., and Jirsa, J. O., 1990, "Behavior of a Structure Strengthened Using Reinforced Concrete Piers," *ACI Structural Journal*, Detroit, Michigan, Vol. 87, No. 5, pp. 557–563.

Bush, T. D., Jones, E. A., and Jirsa, J. O., 1991, "Behavior of RC Frame Strengthening Using Structural Steel Bracing," *Journal of the Structural Engineering Division*, American Society of Civil Engineers, New York, New York.

Bush, T., et al., 1991, "Observations on Two Seismic Strengthening Schemes for Concrete Frames," *Earthquake Spectra*, Earthquake Engineering Research Institute, Oakland, California, Vol. 7, No. 4, pp. 511–527.

Charney, F. A., 1991, "Correlation of the Analytical and Experimental Inelastic Response of a 1/5-Scale Seven-Story Reinforced Concrete Frame-Wall Structure," *Earthquake Resistant Concrete Structures: Inelastic Response and Design*, ACI SP-127, American Concrete Institute, Detroit, Michigan, pp. 261–361.

Corazao, M., and Durrani, A. J., 1989, *Repair and Strengthening of Beam-to-Column Connections Subjected to Earthquake Loading*, Report NCEER-89-0013, National Center for Earthquake Engineering Research, State University of New York at Buffalo, New York.

Corley, G., 1996, "Northridge Earthquake of January 17, 1994 Reconnaissance Report—Concrete Parking Structures," *Earthquake Spectra*, EERI Publication 95-03/2, Earthquake Engineering Research Institute, Oakland, California.

CRC, 1991, *CRC Handbook on Nondestructive Testing of Concrete*, edited by Malhotra, V. M., and Carino, N. J., Boca Raton, Florida.

CRSI, 1981, *Evaluation of Reinforcing Steel Systems in Old Reinforced Concrete Structures*, Concrete Reinforcing Steel Institute, Chicago, Illinois.

Durrani, A. J., Du, Y., and Luo, Y. H., 1995, "Seismic Resistance of Nonductile Slab-Column Connections in Existing Flat-Slab Buildings," *ACI Structural Journal*, American Concrete Institute, Detroit, Michigan, Vol. 92, No. 4, pp. 479–487.

Ehsani, M., and Saadatmanesh, H., 1994, "Out-of-Plane Strength of Masonry Walls Retrofitted with Fiber Composites," *Proceedings*, NCEER Workshop on Seismic Response of Masonry Infills, Report No. NCEER-94-0004, San Francisco, California, National Center for Earthquake Engineering Research, State University of New York at Buffalo, New York, pp. 115-120.

Fiorato, A. E., Sozen, M. A., and Gamble, W. L., 1970, *Investigation of the Interaction of Reinforced Concrete Frames with Masonry Filler Walls*, Report UILU Structural Research Series 370, University of Illinois, Urbana, Illinois.

French, C., and Moehle, J. P., 1991, "Effect of Floor Slab on Behavior of Beam-Slab-Column Connections," *Design of Beam-Column Joints for Seismic Resistance*, Special Publication No. 123, American Concrete Institute, Detroit, Michigan.

Gavrilovic, P., and Sendova, V., 1992, "Experimental and Analytical Studies of Infill Walls in Reinforced Concrete Structures," *Proceedings of the Tenth World Conference on Earthquake Engineering*, A. A. Balkema, Rotterdam, The Netherlands, Vol. 6, pp. 3309–3315.

Gentry, T. R., and Wight, J. K., 1994, "Wide Beam-Column Connections under Earthquake-Type Loading," *Earthquake Spectra*, Earthquake Engineering Research Institute, Oakland, California, Vol. 10, No. 4, pp. 675–703.

Goel, S., and Lee, H., 1990, "Seismic Strengthening of RC Structures by Ductile Steel Bracing System," *Proceedings, Fourth U.S. National Conference on Earthquake Engineering*, Earthquake Engineering Research Institute, Oakland, California, Vol. 13, pp. 323–331.

Goel, S. C., and Masri, A.C., 1994, *Seismic Strengthening of R/C Slab-Column Frames with Steel Bracing: Repair and Rehabilitation Research for Seismic Resistant Structures*, National Science Foundation, Washington, D.C., Preliminary Report.

Goto, T., and Adachi, H., 1987, "Experimental Study of Strengthening Effect on Reinforced Concrete Shear Wall Thickened with Retrofit Wall," *Workshop on Repair and Retrofit of Existing Structures*, US-Japan Panel on Wind and Seismic Effects, Gaithersburg, Maryland.

Graf, W. P., and Mehrain, M., 1992, "Analysis and Testing of a Flat Slab Concrete Building," *Tenth World Conference on Earthquake Engineering*, Madrid, Spain, Vol. 6.

Hawkins, N. M., 1980, "Lateral Load Design Considerations for Flat Plate Structures," *Proceedings, CSCE-American Society of Civil Engineers-ACI-CEB International Symposium on Nonlinear Design of Concrete Structures*, University of Waterloo Press, Ontario, Canada, pp. 581–613.

Hayashi, T., Niwa, H., and Fukuhara, M., 1980, "The Strengthening Methods of the Existing Reinforced Concrete Buildings," *Proceedings, The Seventh World Conference on Earthquake Engineering*, Istanbul, Turkey.

Hwang, S.-J., and Moehle, J. P., 1993, *An Experimental Study of Flat-Plate Structures Under Vertical and Lateral Loads*, Report No. UCB/EERC-93/03, Earthquake Engineering Research Center, University of California at Berkeley, California.

Iglesias, J., 1987, "Repairing and Strengthening of Reinforced Concrete Buildings Damaged in the 1985 Mexico City Earthquake," *The Mexico Earthquake of 1985—Factors Involved and Lessons Learned*, edited by Cassaro, M. A., and Martinez Romero, E., American Society of Civil Engineers, New York, New York, pp. 426–439.

Ishizuka, T., and Hawkins, N., 1987, *Effect of Bond Deterioration on the Seismic Response of Reinforced and Partially Prestressed Concrete and Ductile Moment Resistant Frames*, Report SM 87-2, Department of Civil Engineering, University of Washington, Seattle, Washington.

Jirsa, J. O., and Kreger, M. E., 1989, "Recent Research on Repair and Strengthening of Reinforced Concrete Structures," *Seismic Engineering: Research and Practice*, Structures Congress, San Francisco, California.

Jirsa, J., et al., 1989, "Repairing and Strengthening Damaged Reinforced Concrete Buildings," Lessons Learned from the 1985 Mexico Earthquake, Earthquake Engineering Research Institute, Oakland, California, pp. 175–180.

Joh, O., Goto, Y., and Shibata, T., 1991, "Behavior of Reinforced Concrete Beam-Column Joints with Eccentricity," *Design of Beam-Column Joints for Seismic Resistance*, edited by Jirsa, James O., American Concrete Institute, Detroit, Michigan.

Kahn, L., and Hanson, R., 1979, "Infilled Walls for Earthquake Strengthening," *Journal of the Structural Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 105, No. ST2, pp. 283–296.

Katsumata, H., Kobatake, Y., and Takeda, T., 1988, "A Study on Strengthening with Carbon Fiber for Earthquake-Resistant Capacity of Existing Reinforced Concrete Columns," *Proceedings*, Ninth World Conference on Earthquake Engineering, Tokyo, Japan, Vol. 7, pp. 517–522.

Kitayama, K., Otani, S., and Aoyama, H., 1991, "Development of Design Criteria for Reinforced Concrete Interior Beam-Column Joints," *Design of Beam-Column Joints for Seismic Resistance*, American Concrete Institute, Detroit, Michigan.

Klingner, R. E., and Bertero, V. V., 1978, "Earthquake Resistance of Infilled Frames," *Journal of the Structural Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 104, No. ST6, Procedure Paper 13822, pp. 973–989.

Krause, G. L., and Wight, J. K., 1990, "Strengthening of Reinforced Concrete Frame Structures," *Proceedings*, Fourth U.S. National Conference on Earthquake Engineering, Earthquake Engineering Research Institute, Oakland, California, Vol. 3, pp. 373–382.

Kriviak, G. J., and Scanlon, A., 1987, "Estimation of Compressive Strength of Concrete in Existing Structures Based on In Situ Test Data," *ACI Materials Journal*, American Concrete Institute, Detroit, Michigan.

Kunnath, S. K., Reinhorn, A. M., and Lobo, R. F., 1992, *IDARC version 3.0: A Program for the Inelastic Damage Analysis of Reinforced Concrete Structures*, Report NCEER-92-0022, National Center for Earthquake Engineering Research, State University of New York at Buffalo, New York.

Luke, P. C., Chon, C., and Jirsa, J. O., 1985, *Use of Epoxies for Grouting Reinforced Bar Dowels in Concrete*, Report: PMFSEL 85-2, Ferguson Laboratory, University of Texas at Austin, Texas.

Luo, Y., and Durrani, A. J., 1994, "Seismic Retrofit Strategy for Nonductile Flat Plate Connections," *Proceedings*, Fifth National Conference on Earthquake Engineering, Chicago, Illinois.

Luo, Y., Durrani, A. J., and Conte, J. P., 1994, "Equivalent Frame Analysis of Flat Plate Buildings for Seismic Loading," *Journal of the Structural Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 120, No. 7, pp. 2137–2155.

Lynn, A. C., Moehle, J. P., and Mahin, S. A., 1994, "Evaluation of Reinforced Concrete Columns in Existing Buildings," *EERC News*, Earthquake Engineering Research Center, University of California at Berkeley, California.

Martinez-Cruzado, J. A., Qaisrani, A. N., and Moehle, J. P., 1994, "Post-Tensioned Flat Plate Slab-Column Connections Subjected to Earthquake Loading," *Proceedings*, Fifth National Conference on Earthquake Engineering, Chicago, Illinois; Earthquake Engineering Research Institute, Oakland, California.

Meli, R., and Rodriguez, M., 1979, "Waffle Flat Plate-Column Connections Under Alternating Loads," Bulletin d'Information, Comite Euro-International du Beton, No. 132 bis., Paris, France, pp. 45–52.

Meli, R., 1987, "Evaluation of Performance of Concrete Buildings Damaged by the September 19, 1985 Mexico Earthquake," *The Mexico Earthquake of 1985, Factors Involved and Lessons Learned*, edited by Cassaro, M.A., and Martinez Romero, E., American Society of Civil Engineers, New York, New York, pp. 308–327.

Merriman, M., 1911, *American Civil Engineers' Pocket Book*, John Wiley & Sons, New York, New York.

Miranda, E., and Bertero, V. V., 1991, "Post-Tensioning Technique for Seismic Upgrading of Existing Low-Rise Buildings," *Proceedings*, Fourth U.S. National Conference on Earthquake Engineering; Earthquake Engineering Research Institute, Oakland, California.

Moehle, J. P., Nicoletti, J. P., and Lehman, D. E., 1994, *Review of Seismic Research Results of Existing Buildings*, Report No. SSC 94-03, Seismic Safety Commission, Sacramento, California.

Motooka, J., Ono, A., Adachi, H., and Ohori, K., 1984, "The Effect of Repairing and Strengthening Methods for Reinforced Concrete Shear Walls Damaged by an Earthquake," *Transactions of the Japan Concrete Institute*, Vol. 6.

Nasser, K. W., and Al-Manaseer, A. A., 1987, "Comparison of Nondestructive Testers of Hardened Concrete," *ACI Materials Journal*, American Concrete Institute, Detroit, Michigan.

Otani, S., 1980, "Nonlinear Dynamic Analysis of Reinforced Concrete Building Structures," *Canadian Journal of Civil Engineering*, Vol. 7, No. 2, pp. 333–344.

Otani, S., 1991, "The Architectural Institute of Japan Proposal of Ultimate Strength Design Requirements for RC Buildings with Emphasis on Beam-Column Joints," *Design of Beam-Column Joints for Seismic Resistance*, edited by Jirsa, J. O., American Concrete Institute, Detroit, Michigan.

Otani, S., Kabeyasawa, T., Shiohara, H., and Aoyama, H., 1985, "Analysis of the Full Scale Seven Story Reinforced Concrete Test Structure," *Earthquake Effects on Reinforced Concrete Structures: U.S.-Japan Research*, ACI SP-84, American Concrete Institute, Farmington Hills, Michigan, pp. 203-240.

Pan, A., and Moehle, J. P., 1989, "Lateral Displacement Ductility of Reinforced Concrete Flat Plates," *ACI Structural Journal*, American Concrete Institute, Detroit, Michigan, Vol. 86, No. 3, pp. 250–258.

Pan, A. D., and Moehle, J. P., 1992, "An Experimental Study of Slab-Column Connections," *ACI Structural Journal*, American Concrete Institute, Detroit, Michigan, Vol. 89, No. 6, pp. 626–638.

Park, R., and Thompson, K. J., 1977, "Cyclic Load Tests on Prestressed and Partially Prestressed Beam-Column Joints," *Journal of the Prestressed Concrete Institute*, Chicago, Illinois, Vol. 22, No. 5, pp. 84–110.

Paulay, T., 1971a, "Coupling Beams of Reinforced Concrete Shear Walls," *Journal of the Structural Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 97, No. 3, pp. 843–862.

Paulay, T., 1971b, "Simulated Seismic Loading of Spandrel Beams," *Journal of the Structural Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 97, No. 9, pp. 2407–2419.

Paulay, T., 1986, "The Design of Ductile Reinforced Concrete Structural Walls for Earthquake Resistance," *Earthquake Spectra*, Earthquake Engineering Research Institute, Oakland, California, Vol. 2, No. 4, pp. 783–823.

Pessiki, S. P., Conley, C. H., Gergely, P., and White, R. N., 1990, *Seismic Behavior of Lightly-Reinforced Concrete Column and Beam-Column Joint Details*, Report No. NCEER-90-0014, National Center for Earthquake Engineering Research, State University of New York at Buffalo, New York.

Pinchiera, J. A., and Jirsa, J. O., 1992, "Post-Tensioned Bracing for Seismic Retrofit of RC Frames," *Proceedings*, Tenth World Conference on Earthquake Engineering, Madrid, Spain.

Priestley, N., 1995, *Report of the Fifth U.S. PRESSS Coordinating Meeting*, PRESSS Report 95/1, University of California at San Diego, La Jolla, California.

Priestley, N., et al., 1992, "Column Seismic Retrofit Using Fiberglass/Epoxy Jackets," *Proceedings*, Third NSF Workshop on Bridge Engineering Research in Progress, La Jolla, California, pp. 247–251.

Priestley, M.J.N., Verma, R., and Xiao, Y., 1994, "Seismic Shear Strength of Reinforced Concrete Columns," *Journal of the Structural Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 120, No. 8, pp. 2310–2329.

Qi, X., and Moehle, J. P., 1991, *Displacement Design Approach for RC Structures Subjected to Earthquakes*, Report No. UCB/EERC-91/02, Earthquake Engineering Research Center, University of California at Berkeley, California.

Raffaelle, G. S., and Wight, J. K., 1995, "R/C Eccentric Beam-Column Connections under Earthquake-Type Loading," *ACI Structural Journal*, American Concrete Institute, Detroit, Michigan, Vol. 92, No. 1, pp. 316–323.

Rodriguez, M., and Park, R., 1992, "Lateral Load Response of Reinforced Concrete Columns Strengthened by Jacketing," International Symposium on Earthquake Disaster Prevention, Mexico City, Mexico.

Rodriguez, M., and Park, R., 1991, "Repair and Strengthening of Reinforced Concrete Buildings for Seismic Resistance," *Earthquake Spectra*, Earthquake Engineering Research Institute, Oakland, California, Vol. 7, No. 3, pp. 439–459.

Saatcioglu, M., 1991, "Modeling Hysteretic Force-Deformation Relationships for Reinforced Concrete Elements," *Earthquake Resistant Concrete Structures: Inelastic Response and Design*, ACI SP-127, American Concrete Institute, Farmington Hills, Michigan, pp. 153–198.

Saatcioglu, M., 1995, "Hysteretic Shear Response of Low-Rise Walls," *Concrete Shear in Earthquake*, Elsevier Applied Science, New York, New York, pp. 105–114.

Saiidi, M., 1982, "Hysteresis Models for Reinforced Concrete," *Journal of the Structural Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 108, No. ST5, Procedure Paper 17075, pp. 1077–1087.

Schuller, M., Mehrabi, A. B., Noland, J. L., and Shing, P. B., 1994, "Performance of Masonry-Infilled R/C Frames Under In-Plane Lateral Loads: Experiments," NCEER Workshop on Seismic Response of Masonry Infills, San Francisco, California.

Sheikh, S. A., 1982, "A Comparative Study of Confinement Models," *Journal of the American Concrete Institute*, Detroit, Michigan, Vol. 79, No. 4, Title No. 79-30, pp. 296–306.

Sozen, M. A., 1989, "Earthquake Response of Buildings with Robust Walls," *Proceedings*, Fifth Chilean Conference on Seismology and Earthquake Engineering, Santiago, Chile.

Spacone, E., Ciampi, V., and Filippou, F. C., 1992, *A Beam Element for Seismic Damage Analysis*, Report No. UCB/EERC-92/07, Earthquake Engineering Research Center, University of California at Berkeley, California.

Stoppenhagen, D. R., and Jirsa, J. O., 1987, *Seismic Repair of a Reinforced Concrete Frame Using Encased Columns*, PMFSEL Report No. 87-1, University of Texas at Austin, Texas.

Sugano, S., 1981, "Seismic Strengthening of Existing Reinforced Concrete Buildings in Japan," *Bulletin of the New Zealand National Society for Earthquake Engineering*, Wellington, New Zealand, Vol. 14, No. 4, pp. 209–222.

Thompson, K. J., and Park, R., 1980, "Seismic Response of Partially Prestressed Concrete," *Journal of the Structural Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 106, No. ST8, pp. 1755–1775.

Thomsen, IV, J. H., and Wallace, J. W., 1995, *Displacement Based Design of RC Structural Walls: An Experimental Investigation of Walls with Rectangular and T-Shaped Cross-Sections*, Report: CU/CEE-95/06, Structural Engineering Research Laboratory, Department of Civil and Environmental Engineering, Clarkson University, Potsdam, New York.

Valluvan, R., Kreger, M. E., and Jirsa, J. O., 1992, "Strengthening of Column Splices in Infilled Shear Walls," *Proceedings*, Tenth World Conference on Earthquake Engineering, Madrid, Spain, Vol. 9, pp. 5121–5126.

Valluvan, R., Kreger, M. E., and Jirsa, J. O., 1994, "Seismic Performance of Shear Joints in R/C Frames Retrofitted Using Infill Walls," *Proceedings*, Fifth U.S. National Conference on Earthquake Engineering in Chicago, Illinois; Earthquake Engineering Research Institute, Oakland, California, Vol. III, pp. 655–664.

Vanderbilt, M. D., and Corley, W. G., 1983, "Frame Analysis of Concrete Buildings," *Concrete International*, American Concrete Institute, Detroit, Michigan, pp. 33–43.

Wallace, J. W., and Moehle, J. P., 1992, "Ductility and Detailing Requirements of Bearing Wall Buildings," *Journal of the Structural Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 118, No. 6, pp. 1625–1644.

Wallace, J. W., and Moehle, J. P., 1993, "An Evaluation of Ductility and Detailing Requirements of Bearing Wall Buildings Using Data From the March 3, 1985

Chile Earthquake," *Earthquake Spectra*, Earthquake Engineering Research Institute, Oakland, California, Vol. 9, No. 1, pp. 137–156.

Wallace, J., 1995, "Seismic Design of RC Structural Walls, Part I: New Code Format," *Journal of the Structural Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 121, No. 1, pp. 75–87.

Watabe, M. (Comm. Chair), 1995, *Preliminary Reconnaissance Report of the 1995 Hyogoken-Nanbu Earthquake* (English Edition), Architectural Institute of Japan, Tokyo, Japan.

Wight, J. K., et al., 1996, "On Design Requirements for Reinforced Concrete Structural Walls," *Mete A. Sozen Symposium: A Tribute from his Students*, ACI Special Publication, American Concrete Institute, Farmington Hills, Michigan.

Wood, S. L., Wight, J. K., and Moehle, J. P., 1987, *The 1985 Chile Earthquake, Observations on Earthquake Resistant Construction in Vina Del Mar*, Civil Engineering Studies Report No. SRS 532, University of Illinois at Urbana-Champaign, Illinois.

Wood, S. L., 1991a, "Observed Behavior of Slender Reinforced Concrete Walls Subjected to Cyclic Loading," *Earthquake Resistant Concrete Structures: Inelastic Response and Design*, ACI SP-127, American Concrete Institute, Farmington Hills, Michigan, pp. 453–477.

Wood, S.L., 1991b, "Performance of Reinforced Concrete Buildings During the 1985 Chile Earthquake: Implications for the Design of Structural Walls," *Earthquake Spectra*, Earthquake Engineering Research Center, Oakland, California, Vol. 7, No. 4, pp. 607–638.

Zarnic, R., and Tomazevic, M., 1985, *Study of the Behavior of Masonry Infilled Reinforced Concrete Frames Subjected to Seismic Loading—Part Two*, Report: ZRMK/IKPI-85/02, Institute for Testing and Research in Materials and Structures (Zavod za Raziskavo Materiala in Konstrukcij), Ljubljana, Yugoslavia.

Zarnic, R., Tomazevic, M., and Velechovsky, T., 1986, "Experimental Study of Methods for Repair and Strengthening of Masonry Infilled Reinforced Concrete Frames," *Proceedings*, Eighth European Conference on Earthquake Engineering, Laboratório Nacional de Engineering Civil, Lisbon, Portugal, Vol. 5, 11.1/41–48.

C7. Masonry (Systematic Rehabilitation)

C7.1 Scope

The scope of Chapter 7 is limited to masonry elements that are considered to resist lateral seismic forces as structural members. The chapter includes walls and infill panels subjected to in-plane and out-of-plane forces. Material given is intended to be used directly with the Analysis Procedures prescribed in Chapter 3. All other masonry elements are addressed in Chapters 4 and 11.

C7.2 Historical Perspective

C7.2.1 General

Masonry is the oldest of all construction materials, dating back more than eight millennia to cultures around the globe. Early masonries consisted of stone units with no mortar. The structural action in this form of masonry is much different than that of modern-day clay-unit and concrete masonry, which is found in nearly all existing masonry buildings in the United States, with the exception of some historic buildings that predate the 1850s.

Most masonry buildings in the United States constructed before the 20th century consisted of unreinforced clay-unit masonry. Wythes of brick were commonly tied with brick headers spaced at every sixth or seventh course. Because no other construction material was used for the walls, these building systems represented the first introduction to engineered masonry construction, although seismic considerations were often neglected in the design. Early mortars consisted of no more than lime and sand, which made the shear and tensile strength of the masonry quite weak. In the same era, clay-unit masonry was also used extensively for infills and cladding on steel frame buildings. Though the structural properties of the masonry were ignored in favor of the strong but flexible steel frames, considerable lateral-force resistance was provided by the stiff but brittle masonry, as evidenced by substantial cracking when subjected to earthquake motions.

Following the 1933 Long Beach earthquake, unreinforced masonry (URM) was banned in California, giving rise to reinforced masonry (RM) construction. Today, buildings approaching thirty stories are constructed with stiff, strong, and ductile RM walls designed with limit states concepts. Both hollow-clay and concrete block construction have competed with reinforced concrete and structural steel for the design of commercial, residential, and industrial buildings. In addition, clay-unit masonry remains as the most prevalent material for cladding and veneer on all types of buildings.

In this section, a short treatise on the history of masonry materials is presented to educate the user of these guidelines. Historical summaries are given for:

- clay units
- structural clay tile
- concrete masonry units
- mortar
- reinforced masonry

C7.2.2 Clay Units

Although brick was one of the first products that people manufactured from clay, the era of modern brick began only when extrusion machines were developed. A few bricks were being made by machine in 1833, but the percentage was small until 1870. With the invention of the extrusion or stiff-mud brick-making machine, some manufacturers produced brick containing holes or “cores” running parallel to either the length or the height dimension of the unit. These cores were introduced as an aid to uniform drying of the clay and as a means of reducing the weight of the unit.

The General Assembly of New Jersey passed a law in 1883 to establish brick dimensions at 9-1/2" x 4-1/2" x 2-3/4". In 1889, in the District of Columbia, the ordinance of October 31, 1820 was still being enforced, which fixed a minimal size of brick at 9-1/4" x 4-5/8" x 2-1/4".

In 1929, a report prepared by McBurney and Logwell summarized that 92% of the brick produced in the United States had flat-wise compressive strengths averaging 7,246 psi for both hard and salmon brick. From the distribution data given, approximately 6% of the production classified as 1,250 to 2,500 psi, 20% as 2,225 to 4,500 psi, and 74% as over 4,500 psi. Approximately 40% of the production was 8,000 psi or over in compressive strength.

Solid brick is now defined as a small building unit, solid or cored not in excess of 25%, commonly in the form of a rectangular prism, formed from inorganic, nonmetallic substances, and hardened in its finished shape by heat or chemical action. Brick is also available in larger units with cell or core areas up to 60% of the cross section. Such units are typically used for placement of both vertical and horizontal reinforcement. The term “brick,” when used without a qualifying adjective, is understood to mean such a unit or a collection of such units made from clay or shale hardened by heat.

C7.2.3 Structural Clay Tile

Structural clay tile is a machine-made product first produced in the United States in New Jersey in 1875. Structural clay tiles are characterized by the fact that they are hollow units with parallel cells (hollow spaces). The shape of the unit is controlled by the die through which the clay column is extruded. The ease with which different designs could be produced led to the development of a wide variety of sizes and patterns.

In 1903, the National Fireproofing Corporation of Pittsburgh published a handbook and catalog by Henry L. Hinton, illustrating the products of the company and presenting data for use in the design of segmental and flat arch floors. This catalog is of historical interest, particularly because of the large number of unit designs illustrated. Hundreds of different shapes are shown for use in the construction of tile floor arches, partitions, and walls, and for fireproofing columns, beams, and girders.

Structural clay tile was used extensively during World War I. With lumber in critically short supply, hollow-clay tile was largely relied upon for all types of buildings. Brick and tile were used for the construction of mobilization structures, war housing, defense plants, air fields, and buildings at army and navy bases.

In 1950, structural clay tile was classified under the following types: Structural Clay Load-Bearing Wall Tile, Structural Clay Non-Load-Bearing Tile (partition, furring, and fireproofing), Structural Clay Floor Tile, Structural Clay Facing Tile, and Structural Glazed Facing Tile.

C7.2.4 Concrete Masonry Units

The earliest specification for hollow concrete block was proposed by the National Association of Cement Users in January 1908. The NACU was organized in 1904 and continued under that name until 1913, when it became known as the American Concrete Institute (ACI).

In 1905, the United States government adopted concrete block for its hospitals, warehouses, and barracks in the Panama Canal Zone and the Philippine Islands.

The 1908 specification called for the block in bearing walls to have an average strength of 1000 psi at 28 days with a minimum of 700 psi. Air space was limited to 33% and absorption was to average not more than 15% (with no single block to exceed 22%). Absorption was to be measured on a block placed in a pan of water at least 2" deep. Fine aggregate had to pass a 1/4" mesh sieve; stone or clean-screened gravel was to go through a 3/4" sieve and be refused on a 1/4" sieve. A 1-3-4 semi-wet mix was recommended for exposed bearing walls, and a 1-3-5 mix for a wet cast block. Portland cement mortar was recommended. Transverse, compressive, and absorption tests were required, along with freezing and fire tests when necessary, and the modulus of rupture at 28 days was to average 150. Any expense attending such tests was to be met by the manufacturer of the block.

This first standard specification was adopted in 1910. Two years later, the practice for curing—which until that time had consisted of sprinkling with water for seven days—was revised slightly by the addition of a new method, the use of steam from 100 to 130°C for 48 hours with a subsequent storing of eight days. This recommended practice was the first mention of high-pressure steam curing in block specifications.

In 1916, the absorption rate was changed to 10% at the end of 48 hours. In 1922 came the first specification for a non-load-bearing unit, with a requirement of 300 psi. That same year, the following strengths were suggested: 250, 500, 700, and 1200 psi for non-load-bearing, light-load-bearing, medium-load-bearing, and heavy-load-bearing walls, respectively. The ACI accepted these values as tentative in 1923. The absorption time, however, was shortened from 48 to 24 hours. A similar table, with the elimination of the light-load-bearing unit, was accepted as tentative in 1924, and adopted the following year.

By 1928, more than 80 city building codes had been revised to eliminate practically all of the legal obstacles to the increased use of concrete block. Public works construction by state and local governments had declined steadily until by 1933 it had virtually ceased. In 1933, several government agencies were set up to purchase concrete block. In July 1935, the National Industrial Recovery Act was invalidated by the U.S. Supreme Court, but it had by then performed a valuable service for the concrete block industry. Although business activity in the 1930s was in a constantly deepening trough of despair, lifted only by public building programs, the decade was surprisingly productive in a good many technological areas for the concrete block industry.

C7.2.5 Mortar

The common variety of mortar was made of lime, sand, and water. Details of its preparation varied according to regional customs and individual preferences, but most of these details were well known throughout Europe and America. Sand was added to lime for economy, to prevent shrinkage, and in such quantity that the lime would fill the interstices. If an excess of sand was used, the bond was poor. If too little sand was used, the mortar would shrink and crack.

In ordinary sands, the spaces were from 39% to 40% of the total volume, and in such, 1.0 volume of cementitious paste (cement plus lime) would fill voids of 2.5 volumes of sand. In practice, 1.25 to 2.0 volumes of sand to 1.0 of paste was used. Thus, “pure” lime mortar meant three to five volumes of sand to one measured volume of lime. This gave a plastic mortar that did not crack.

Until about 1890, the standard mortar used for masonry in the United States was a mixture of sand and pure lime (i.e., hydraulic lime) or lime-pozzolon-sand. Massachusetts Hall (1730) at Harvard University and Independence Hall (1734) in Philadelphia were built with lime mortars that were also known as “fact” mortars. These low-strength mortars gave masonry a low modulus of elasticity and, therefore, an ability to absorb considerable strain without inducing high stress. Accordingly, the tendency to crack was reduced, and when cracks did appear, masonry of high lime-content mortar was to a great extent capable of chemical reconstitution, i.e., “autogenous healing.”

After 1819, all masonry used in the construction of the Erie Canal was laid in natural cement mortar. Various sources afford different information about the mortar mix; apparently one part of sand was mixed with two parts of cement. The general practice in New York state in about 1840 was to mix two or three parts of sand to one of cement.

For natural cements, the proportion of sand to cement by measurement usually did not exceed three to one, and for piers and first-class work a ratio of two to one was used. Portland cement mortar commonly contained four parts of sand to one of cement for ordinary mortar, and three to one for first-class mortar. For work under water, not more than two parts of sand to one of cement were used. When cheaper mortars than these were desired, it was considered better to add lime to the mortar than more sand. Cement mortars were introduced about 1880. Joints of cement mortar were strong and unyielding because of the cement; they were appropriate for bonding to modern bricks and concrete blocks.

C7.2.6 Reinforced Masonry

Reinforced brick masonry was first used by Marc Isambard Brunel in 1825, in the building of the Thames Tunnel in England (Plummer and Blume, 1953). Reinforced brick masonry was used by many builders during that century; however, these builders were individuals who had a feel for materials and built their structures based upon their experience, more as an art than from a rational design. Prior to 1880, a few attempts were made to develop design formulas. However, the performance of composite steel and masonry flexural members was not clearly understood at that time, and many investigators have attributed the strength of the construction primarily to the adhesive properties of the masonry. In fact, most of the early tests were designed to demonstrate the increased strength obtainable through the use of a new Portland cement in mortar, instead of the hydraulic limes and natural cements formerly used.

In the United States, Hugo Filippi, C.E. built and tested reinforced brick masonry beams in 1913. Later in 1919, L.J. Mensch, C.E. of Chicago also tested reinforced brick beams in which the reinforcement was placed in a bed of mortar below the brick masonry. However, the data from these tests and others were never published and there was little, if any, exchange of information among those interested in the subject.

In 1923, the Public Works Department of the Government of India published Technical Paper #38, a comprehensive report of extensive tests of reinforced brick masonry structures extending over a period of about two years. A total of 282 specimens were tested, including reinforced brick masonry slabs of various thicknesses, reinforced brick beams, both reinforced and unreinforced columns, and reinforced brick arches. These tests appeared to be the first organized research on reinforced brick masonry; the data provided answers to many questions regarding this type of construction. This research may therefore be considered as marking the initial stage of the modern development of reinforced brick masonry.

The idea of using cement-sand grout instead of bonding brick headers to bind brick wythes or tiers together, and inserting reinforcing steel in the grout space for tensile and shearing resistance, was developed for practical and sound engineering use in southern California beginning about 1935. Since then, thousands of tests have been conducted on full-size beams, slabs, and walls, from which sound engineering design criteria have been established and incorporated into building codes throughout the United States.

C7.3 Material Properties and Condition Assessment

C7.3.1 General

The term “masonry” is used to define the composite of units, mortar, and possibly grout and/or reinforcement. Whereas there are specifications to control the manufacture of each of the constituent materials, the most basic engineering properties to consider for analysis of a building system are those representing the composite. Thus, permissible values are given in this section for compressive strength and elastic modulus of the masonry assemblage, flexural tensile strength at the unit-mortar interface, and shear strength and shear modulus of vertical components such as piers, panels, and walls. These mechanical properties will be relied on for estimating stiffness and strength of masonry wall and infill components.

C7.3.2 Properties of In-Place Materials

C7.3.2.1 Masonry Compressive Strength

Three options are given for measuring expected masonry compressive strength. The first two methods rely on testing of either extracted or rebuilt masonry prisms in a laboratory. The third method measures strength in situ by inserting a pair of flat jacks in an existing masonry wall.

For the first method, sample test prisms are extracted from a masonry component and transported to a laboratory. The test prisms are subjected to vertical compressive stress until the peak strength is reached. The prism height should be at least twice its thickness, contain at least two bed joints, and be a minimum of 15 inches high. The advantage of this method is that an actual prism can be tested under controlled laboratory conditions. In addition, strains can be monitored to infer the elastic modulus (see Section C7.3.2.2). The disadvantages are that the compressive strength might be reduced during extraction, and the number of test specimens is limited because of the cost of both the extraction and the repair of the wall.

The second method requires test prisms to be fabricated from actual masonry units that are extracted from an existing masonry component. A chemical analysis of the mortar is required so that mix proportions can be simulated, and the mortar can be recreated. The advantage of this method is the same as for the first method. The disadvantage is that long-term creep, moisture, and temperature effects cannot be simulated.

The third method consists of cutting slots in two mortar bed joints, four to six courses apart, so that thin, hydraulic flat jacks can be inserted and pressurized. The portion of the masonry between the jacks is subjected to a state of vertical compressive stress. The jacks are stressed until the strength of the masonry is reached. For masonry that is relatively weak, softening can be observed by a reduction in slope of the stress-strain curve, and compressive strength can be inferred. The advantage of this method is that it is nondestructive and the strength is measured in situ. In addition, the test can be done in concert with other tests done to measure elastic modulus and in situ compressive stress. The disadvantage is that typical flat jacks may not be able to reach the high pressures needed to approach the ultimate strength of the masonry in compression.

As an alternative to the test methods given in the *Guidelines*, the expected masonry compressive strength may be deduced from a nominal value prescribed by the Masonry Standards Joint Committee specification for new construction (MSJC, 1995) knowing the unit strength and mortar type (Specification Table 1 for clay-unit masonry and Table 2 for concrete masonry). Tests of extracted masonry units may be done to ascertain the unit strength, or conservative estimates of unit strength can be assumed for use with the MSJC tables. Likewise, mortar type can be evaluated experimentally or assumed. The MSJC table values are based on data from masonry constructed after the 1950s and are only applicable to this period of construction. Many of the earlier mortars were lime-based rather than cement-based as assumed with these table values. Furthermore, earlier mortars were classified with a different nomenclature than given in these tables, making direct relations difficult. Therefore, the unit-strength procedure using the MSJC tables should only be used for masonry constructed after 1960. Expected masonry strength should be determined by multiplying Table 1 values by a factor of 2.0 or Table 2 values by a factor of 1.5. These approximate factors are based on estimated ratios between expected and lower bound compressive strengths, as well as on correction factors for clay brick and concrete block prisms.

Default values of compressive strength are set at very low stresses to reflect an absolute lower bound. Masonry in poor condition is given a strength equal to one-third that for masonry in good condition, to reflect the influence of mortar deterioration and unit cracking on compressive strength.

C7.3.2.2 Masonry Elastic Modulus in Compression

The elastic modulus of masonry in compression can be measured by one of two methods. Each method measures vertical strain between two gage points to infer strain, and thus elastic modulus. The first method consists of extracting a test prism from an existing wall; the second method utilizes a pair of flat jacks to subject an in situ portion of masonry to vertical compressive stress.

The extracted prism method is essentially the same as for the compressive strength test, with the difference that dial gages or electronic displacement transducers are fixed on the test prism to measure strain between two gage points.

The flat-jack method is done in the same way as for the compressive strength test, with the difference that the jacks are pressured to less than half the masonry strength. Vertical contractions of the compression field between the two jacks are measured with a mechanical dial gauge or electronic displacement transducer. Strain is then determined by dividing measured distortion by the length between gauges. Using correction factors for shape and stiffness of flat jacks, vertical compressive stress is inferred from measured hydraulic pressure. The elastic modulus, E_{me}, is calculated as the slope of the stress-strain curve between 5% and 33% of the estimated masonry ultimate compressive strength.

The flat-jack method has been shown to be accurate within 10%, based on correlations between test values and measured elastic moduli of test prisms (Epperson and Abrams, 1989; Noland et al., 1987). A case study using this method is presented by Kariotis and Ngheim (1995). An available standard is the *Standard Test Method for In-Situ Elastic Modulus within Solid Unit Masonry Estimated Using Flat Jack Measurements*, ASTM C 1197.

Default values of elastic modulus shall be based on a coefficient of 550 times the expected masonry compressive strength. This coefficient is set lower than previous values given in the *Uniform Building Code* to compensate for larger values of expected strength.

C7.3.2.3 Masonry Flexural Tensile Strength

Although the flexural tensile strength of older brick masonry walls constructed with lime mortars may often be neglected, the tensile strength of newer concrete and clay-unit masonry walls can result in appreciable flexural strengths. Therefore, guidelines for measuring flexural tensile strength in situ or from extracted specimens are given in this section.

Masonry flexural tensile strength can be measured using a device known as a bond wrench, which clamps onto the top course of a test specimen and applies a weak-axis bending moment until the top masonry unit snaps off. Flexural tensile stress is inferred by dividing the moment capacity by the section modulus of the wall section. The test can be done on test specimens extracted from an existing wall, or in situ on a portion of masonry that has been isolated by cutting vertical slots on either side of the test portion. Alternatively, flexural tension stress can be measured by bending extracted portions of a masonry wall across a simply supported span.

For the field test, two adjacent units of a running bond pattern are removed so that a clamp may be inserted. Single masonry units above and below the removed units are subjected to an out-of-plane moment using a calibrated torque wrench. Mortar head joints on either sides of the tested units are removed to isolate the test units. The laboratory test is done in much the same manner on specimens that are cut from a wall. Test prisms should be at least two units in height, and one unit long, or a minimum of four inches. Both methods involve substantial repair of the existing wall. An available standard for the laboratory method is *Standard Test Methods for Masonry Bond Wrench Testing*, ASTM C 1072. No standards exist on the field bond wrench test; however, this ASTM standard should suffice.

The third method consists of extracting sample panels or prisms from an existing masonry wall, and subjecting them to minor-axis bending with either a third-point loading or a uniform load distribution with an airbag. Flexural tensile strength is determined by dividing the maximum applied moment by the section modulus of the masonry section. *Standard Test Methods for Masonry Flexural Tension Stress*, ASTM E 518, is available; however, ASTM does not recommend this method for determination of design stresses.

For all three of these methods, the bonding of the test unit to the mortar is sensitive to any disturbances that are incurred during specimen removal. The confidence level can be low because the scatter of data for flexural bond strength can be high, and the number of test samples is limited because of cost and the disturbance concerns.

These test methods are intended for out-of-plane strength of unreinforced masonry walls. For in-plane bending, flexural stress gradients across the section width are much lower than for out-of-plane bending. Thus, data from tests described in this section should not ideally be used for in-plane bending. However, in lieu of data on in-plane tensile strength, out-of-plane strength values may be substituted.

Default values for flexural tensile strength are set low even for masonry in good condition, because of the dependence of the unit-mortar bonding on the tensile strength. This bonding can be highly variable, depending on the relative absorption of the unit and the water retentivity of the mortar, the presence and type of cement used in the mortar, the previous loading history, and the condition of the mortar. For masonry in poor condition, a zero value of tensile strength is prescribed.

C7.3.2.4 Masonry Shear Strength

Expected shear strength of URM components can be inferred from in situ measurements of bed-joint shear strength using the in-place shear test. The nondestructive test measures the in situ shear strength between a clay masonry unit and the mortar bed joints above and below the unit. A small hydraulic jack is placed in a void left by removal of a masonry unit immediately adjacent to the test unit. The head joint on the opposite face of the test unit is removed to isolate the test unit so that it may be displaced horizontally when pushed.

A horizontal force is applied to the test unit until it starts to slide. Shear strength is then inferred as the measured force divided by the area of the bed joints above and below the masonry unit. The estimated vertical compressive stress at the test location is subtracted from this value to give the bed joint shear stress, v_{to} (Equation 7-2), assuming a coefficient of friction equal to 1.0. Because expected values of wall shear strength are to be used, the 50th percentile value, v_t, is used as the index value.

The method is limited to tests of the face wythe. When the test unit is pushed, resistance is provided across not only the bed-joint shear planes, but also the collar-joint shear plane. Because seismic shear is not transferred across the collar joint in a multiwythe masonry wall, the estimated shear resistance of the collar joint must be deducted from the test values. This is done by multiplying the v_{te} term by 0.75 in Equation 7-1, which is the ratio of the areas of the top and bottom bed joints to the sum of the areas of the bed and collar joints for a typical clay unit. If it is known that the collar joint is not present, or is in very poor condition, the 0.75 factor may be waived.

The effect of friction at the particular location of the masonry element being evaluated is included by increasing the bed-joint shear capacity by the addition of the term “P/A” in Equation 7-1. The sum is then multiplied by a reduction factor equal to 0.75, and divided by 1.5 to convert it to an average stress for use with walls of a rectangular cross section.

The in-place shear test was developed solely for solid clay-unit masonry. However, the test method has been used for single-wythe hollow concrete block masonry. As for the conventional method with brick masonry, a single unit is removed adjacent to a test unit as well as the opposite mortar head joint. The maximum horizontal force needed to move the block is divided by the total area of the bed joint mortar above and below the test unit and the total grouted area. The term v_{to} is obtained by subtracting the apparent vertical compressive stress from this ratio as given in Equation 7-2. If the shear capacity of the masonry exceeds that of the loading equipment, the test may be run on one-half the length of a block. In such case, the mortar bed joints along one-half the length of the block are removed.

An alternate in-place shear test method is to simultaneously apply a vertical compressive stress, using hydraulic flat jacks placed in the bed joints above and below the test brick, while shearing the test brick. In-place shear tests are done at various levels of vertical compressive stress so that values of cohesion and frictional coefficients can be inferred.

The available standard *In-Place Masonry Shear Tests* (UBC Standard 21-6), is referenced in the 1994 *Uniform Code for Building Conservation* (ICBO, 1994), Appendix Chapter 1, Sections A106(c)3 and A107(b).

Default values for shear strength of URM are provided, ranging from 27 psi for good condition to 13 psi for poor condition. If in-place shear tests are done, the upper bound of v_{me} by Equation 7-1 is 37 psi for a zero vertical compressive stress when the 100 psi limit on v_{te} is considered. Thus, a 37% increase in strength is possible if testing is done and the masonry is considered to be in good condition. Default values for shear strength of poor masonry are large relative to values for masonry in good condition (1:2), because frictional shear can be developed even when mortar or units are deteriorated.

Shear strength of reinforced masonry (RM) cannot be expressed in terms of the bed-joint shear stress because of the influence of the vertical and horizontal reinforcement on shear strength. There are no in situ methods for measuring shear strength of existing RM walls. Equations given for shear strength in BSSC (1995) must be relied on. Ideally, the theory of mechanics of materials does not change with age, and the same strength equations should apply for existing or new construction. However, care should be taken to ensure that the condition of the existing masonry components is comparable to that of newly constructed elements. This assessment should include a review of reinforcing details as well as the general condition of the masonry (see Section 7.3.3).

C7.3.2.5 Masonry Shear Modulus

Laboratory tests of URM shear walls (Epperson and Abrams, 1989; Abrams and Shah, 1992) have found that the shear modulus of masonry does approach the value of 0.4 times the elastic modulus in compression, as given by the theory of elasticity for isotropic, elastic members. This value is limited to elastic, uncracked behavior of the masonry. After cracking, the shear stiffness is known to reduce substantially as sliding along bed joints develops or as diagonal tension cracks open. Because these nonlinear effects cannot be related to the elastic modulus in compression, the $0.4E_m$ value is only appropriate for uncracked masonry. Shear stiffness of post-cracked masonry can be taken as a fraction of the initial shear stiffness. Test data by Atkinson et al. (1989) provide estimates of shear stiffness based on a frictional mechanism along bed joints.

C7.3.2.6 Strength and Modulus of Reinforcing Steel

The expected strength of reinforcing bars can be best determined from tension tests of samples taken from the building. If available, mill test data for the reinforcing steel used in the building may be substituted.

Default values of yield strength are given to be the same as for reinforcing bars in reinforced concrete (see Section 6.3.2.5).

C7.3.2.7 Location and Minimum Number of Tests

The required number of tests have been established based on theories of statistical sampling, and past experience.

C7.3.3 Condition Assessment

The goals of a condition assessment are:

- 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 continuity of load paths between components, elements, and systems
- To review other conditions, such as neighboring party walls and buildings, presence of nonstructural

components, and limitations for rehabilitation, that may influence building performance

- To formulate a basis for selecting a knowledge factor

The physical condition of existing components and elements, and their connections, should be examined for deterioration of masonry units, mortars, grouts, and reinforcement. Deterioration may include environmental effects (e.g., fire damage, chemical attack, freeze/thaw damage) or past/current loading effects (e.g., overload, damage from past earthquakes, cracking). Masonry construction is also susceptible to expansion and contraction due to thermal and moisture conditions.

A condition assessment should examine configuration problems such as discontinuous reinforcement patterns, unequal alignment of components, and inadequate connections between walls and foundation.

The scope of a condition assessment shall include an investigation of primary and secondary structural elements and components. Although masonry veneer is not part of the structural system, the condition and attachment of the veneer should be examined. Substantial damage to masonry veneer has been observed in numerous earthquakes (Klingner, ed., 1994). Rehabilitation measures should be undertaken to mitigate damage to veneer. However, since the veneer is not part of the structural system, such measures will not involve the Systematic Rehabilitation procedures prescribed in Chapter 7. 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 are invaluable to the understanding of load paths and the ability of components to resist and transfer these loads.

Destructive or nondestructive test methods may be necessary to examine the interior portions of a masonry structural component. Local removal of sheathing or coatings on masonry wall surfaces may need to be done to expose connections between the masonry and adjoining components. The number of such examinations will vary with the complexity and availability of construction drawings.

C7.3.3.1 Visual Examination

Visual observations are simple and generally inexpensive, and can detect obvious condition states in the masonry materials and quality of construction. Configuration problems can quickly be identified with direct visual inspection. The continuity of load paths can be determined through viewing of components and connection condition. Visual inspection can determine the need for other test methods to quantify the presence and degree of deterioration.

The process of establishing component properties should start with obtaining construction documents. Preliminary review of these documents should be done to identify primary gravity- and lateral-load-carrying elements, systems, components, and connections. In the absence of a complete set of building drawings, a thorough inspection of the building should be done to identify all load-bearing systems, and an as-built set of drawings should be made.

If coverings or other obstructions exist, indirect visual inspection can be done through use of drilled holes and a fiberscope.

C7.3.3.2 Nondestructive Tests

Four tests are recommended to assess the relative condition of masonry components: ultrasonic pulse velocity, mechanical pulse velocity, impact echo, and radiography. Merits and limitations of each method are described in this section. Further information can be found in Abrams and Matthys (1991).

A. Ultrasonic Pulse Velocity

Measurement of the velocity of ultrasonic pulses through a wall can detect variations in the density and modulus of masonry materials as well as the presence of cracks and discontinuities. Transmission times for pulses traveling through a wall (direct method) or between two points on the same side of a wall (indirect method) are measured and used to infer wave velocity.

Test equipment with wave frequencies in the range of 50 kHz has been shown to be appropriate for masonry walls. Use of equipment with higher-frequency waves is not recommended because the short wave length and high attenuation are not consistent with typical dimensions of masonry units.

Test locations should be sufficiently close to identify zones with different properties. Contour maps of direct transmission wave velocities can be constructed to assess the overall homogeneity of a wall elevation. For indirect test data, vertical or horizontal distance can be plotted versus travel time to identify changes in wave velocity (slope of the curve). Abrupt changes in slope will identify locations of cracks or flaws.

Ultrasonic methods are not applicable for masonry of poor quality or low modulus, or with many flaws and cracks. The method is sensitive to surface condition, the coupling material used between the transducer or receiver and the brick, and the pressure applied to the transducer.

The use of ultrasonic pulse velocity methods with masonry walls has been researched extensively (Calvi, 1988; Epperson and Abrams, 1989; Kingsley et al., 1987). A standard for the use of ultrasonic methods for masonry is currently under development in Europe with RILEM Committee 76LUM.

B. Mechanical Pulse Velocity

The mechanical pulse velocity test consists of impacting a wall with a hammer blow and measuring the travel time of a sonic wave across a specified gage distance. An impact hammer is equipped with a load cell or accelerometer to detect the time of impact. A distant accelerometer is fixed to a wall to detect the arrival time of the pulse. Wave velocity is determined by dividing the gage length by the travel time. The form and duration of the generated wave can be varied by changing the material on the hammer cap.

The generated pulse has a lower frequency and higher energy content than an ultrasonic pulse, resulting in longer travel distances, and less sensitivity to small variations in masonry properties and minor cracking. The mechanical pulse method should be used in lieu of the ultrasonic pulse method when overall mean properties of a large portion of masonry are of interest.

The use of mechanical pulse velocity measurements for masonry condition assessments has been confirmed through research (Epperson and Abrams, 1989; Kingsley et al., 1987). Although no standard exists for mechanical pulse velocity tests with masonry, a standard for concrete materials does exist, which may be referenced: *Test Method for Pulse Velocity through Concrete (10-150 kHz range)*, ASTM C 597.

C. Impact Echo

The impact-echo technique can be useful for nondestructive determination of the location of void areas within grouted reinforced walls (Sansalone and Carino, 1988). Commercial devices are available or systems can be assembled using available electronic components. Since this technique cannot distinguish between a shrinkage crack at the grout-unit interface and a complete void in the grout, drilling of small holes in the bed joint or examination using an optical borescope should be performed to verify the exact condition.

D. Radiography

A number of commercial devices exist that can be used to identify the location of reinforcing steel in masonry walls. They are also useful for locating bed-joint reinforcing steel, masonry ties and anchors, and conduits and pipes. The better devices can locate a No. 6 bar at depths up to approximately six inches; however, this means that for a 12-inch-thick concrete masonry wall, a bar located off-center cannot be found when access is limited to only one side of the wall. These devices are not able to locate or determine the length of reinforcing bar splices in walls for most cases. They work best for identifying the location of single isolated bars, and become less useful when congestion of reinforcing bars increases.

C7.3.3.3 Supplemental Tests

A. Surface Hardness

The surface hardness of exterior-wythe masonry can be evaluated using the Schmidt rebound hammer. Research has shown that the technique is sensitive to differences in masonry strength, but cannot by itself be used to determine absolute strength. A Type N hammer (5000 lb.) is recommended for normal-strength masonry, while a Type L hammer (1600 lb.) is recommended for lower-strength masonry. Impacts at the same test location should be continued until consistent readings are obtained, because surface roughness can affect initial readings.

The method is limited to tests of only the surface wythe. Tuckpointing may influence readings and the method is not sensitive to cracks.

Measurement of surface hardness for masonry walls has been studied (Noland et al., 1987).

B. Vertical Compressive Stress

In situ vertical compressive stress resisted by the masonry can be measured using a thin hydraulic flat jack that is inserted into a removed mortar bed joint. Pressure in the flat jack is increased until distortions in the brickwork are reduced to the pre-cut condition. Existing vertical compressive stress is inferred from the jack hydraulic pressure, using correction factors for the shape and stiffness of the flat jack.

The method is useful for measurement of gravity load distribution, flexural stresses in out-of-plane walls, and stresses in masonry veneer walls that are compressed by a surrounding concrete frame. The test is limited to only the face wythe of masonry.

Not less than three tests should be done for each section of the building for which it is desired to measure in situ vertical stress. The number and location of tests should be determined based on the building configuration, and the likelihood of overstress conditions.

C. Diagonal Compression Test

A square panel of masonry is subjected to a compressive force applied at two opposite corners along a diagonal until the panel cracks. Shear strength is inferred from the measured diagonal compressive force based on a theoretical distribution of shear and normal stress for a homogeneous and elastic continuum. Using the same theory, shear modulus is inferred from measured diagonal compressive stress and strain.

Extrapolation of the test data to actual masonry walls is difficult because the ratio of shear to normal stress is fixed at a constant ratio of 1.0 for the test specimens. Also, the distribution of shear and normal stresses across a bed joint may not be as uniform for a test specimen as for an actual wall. Lastly, any redistribution of stresses after the first cracking will not be represented with the theoretical stress distributions. Thus, the test data cannot be useful to predict nonlinear behavior.

If the size of the masonry units relative to the panel dimension is large, masonry properties will be not continuous, but discrete. Test panels should be a minimum of four feet square. The high cost and disruption of extracting a number of panels this size may be impractical.

A standard is available, titled *Standard Test Method for Masonry Diagonal Compression*, ASTM E 519.

D. Large-Scale Load Tests

Large-scale destructive tests may be done on portions of a masonry component or element to (1) increase the confidence level on overall structural properties, (2) obtain performance data on archaic building materials and construction materials, (3) quantify effects of complex edge and boundary conditions around openings and two-way spanning, and (4) verify or calibrate analytical models. Large-scale load tests do not necessarily have to be run to the ultimate limit state. They may have value for simply demonstrating structural integrity up to some specific Performance Level.

Out-of-plane strength and behavior of masonry walls can be determined with air-bag tests. Behavior of test panels incorporating connections and edge details can be determined from such a test, in addition to flexural and arching properties of a solid or perforated wall.

Strength and deformation capacity under in-plane lateral forces can be determined by loading an individual portion of wall that is cut free of the surrounding masonry. Loading actuators are reacted against adjacent and stronger portions of masonry. Such testing is particularly useful when the wall is composed of different materials that cannot be evaluated by testing an individual unit of an individual wythe.

Visual and nondestructive surveys should be used to identify locations for test samples.

Standards for laboratory test methods are published by ASTM. Procedures for removal and transportation of masonry samples are given in *Evaluation of Structural Properties of Masonry in Existing Buildings*, NBS Building Science Series 62, U.S. Department of Commerce.

Large-scale tests are expensive and limited to a single or few samples. They may result in considerable local damage and may require substantial reconstruction near the sample location. Test data must be extrapolated to the remainder of the system based on a low confidence level.

C7.3.4 Knowledge (κ) Factor

The level of knowledge of a particular masonry structure may conform to either a minimum level or an enhanced comprehensive level. As noted in Section 2.7.2, knowledge factors, κ, are assigned equal to 0.75 and 1.00 for these two levels. The Linear Static Procedure (LSP) of Chapter 3 may be used with either knowledge level, but the Nonlinear Static Procedure (NSP) is limited to a κ factor equal to 1.0.

The basic distinction between the two levels of knowledge is whether or not in situ tests of masonry materials are done. For the minimum level, a visual examination of the structure is required per Section 7.3.3.1; however, in-place testing is not necessary. Thus, the LSP may be used with the default values of material strengths as specified in Section 7.3.2. For the comprehensive level of knowledge, some in situ material testing is required in addition to the nondestructive testing for condition assessment noted in Section 7.3.3.2. These tests include determination of masonry compressive strengths using one of the methods prescribed in Section 7.3.2.1 for both unreinforced and reinforced masonry. For unreinforced masonry only, in-place shear strength tests must be done in accordance with Section 7.3.2.4. For reinforced masonry only, tensile strengths of reinforcing bars must be determined in accordance with Section 7.3.2.6.

Even for the comprehensive level of knowledge, in situ tests of masonry flexural tensile strength or elastic modulus are not required. This is because tensile strength should be quite low and somewhat similar to the default values as given in Section 7.3.2.3. Similarly, test data for elastic modulus can have a large scatter and not differ from the approximate value given in Section 7.3.2.2 (550 times the masonry expected compressive strength).

C7.4 Engineering Properties of Masonry Walls

Masonry building systems are composed largely of walls. Masonry walls may be divided between structural walls—such as bearing or shear walls—and nonstructural walls, such as partition walls, cladding, veneer, infills, and parapets. Engineering properties given in Section 7.4 apply only to structural walls.

Masonry bearing walls support floor and roof gravity loads, and may or may not be shear walls. Conversely, masonry shear walls resist lateral seismic forces, and may or may not be bearing walls. If a wall is part of the lateral-force-resisting system, it is considered as a primary element. If the wall supports only gravity loads and must remain stable under lateral sway, it is considered as a secondary element. All other masonry walls are excluded from Section 7.4.

C7.4.1 Types of Masonry Walls

Structural masonry walls are classified into three fundamental types: existing, new, and enhanced. Guidelines for determining structural properties of masonry walls reference current standards, which are different for existing and new walls. In addition, the *Guidelines* provide specific recommendations on minimum requirements for enhancement of existing walls so that their structural properties may be considered the same as those of new or existing elements or components.

Rehabilitated buildings typically consist of lateral-force-resisting systems that comprise a combination of different materials. An existing unreinforced masonry building might be strengthened by adding braced steel frames, or conversely, a new reinforced masonry wall might be added to stiffen a flexible steel frame. Existing masonry walls might be enhanced with shotcrete or surface coatings, reinforced or prestressed cores, grout injections, or repointing, or by changing the size of openings. The engineering properties given in Section 7.4 are applicable to building systems with existing, new, or enhanced masonry walls that combine to rehabilitate a building system.

Stiffness assumptions, strength criteria, and acceptable deflections for various limit states as described in Sections 7.4.2 through 7.4.5 are common for existing, new, or enhanced masonry walls. Principles of mechanics are the same despite the age of a masonry wall. Physically, there should be no difference in stiffness assumptions, strength criteria, or inelastic behavior for existing, newly constructed, or enhanced walls. Thus, guidelines on determining engineering properties for each of the three fundamental wall types are expressed in common in these sections.

In Sections 7.4.2 through 7.4.5, walls are grouped in terms of how they respond to lateral forces. Unreinforced walls are presented first, followed by reinforced walls, because the behavior of each type of wall is distinctively different. Furthermore, walls subjected to in-plane lateral forces are separated from walls subjected to out-of-plane forces because their stiffnesses, strengths, and acceptable deformations vary widely.

C7.4.1.1 Existing Masonry Walls

Existing masonry walls will have a significant influence on the lateral strength and drift of a building system. Certain masonry walls may have a brittle character, and partial or complete removal may improve the overall energy dissipation capabilities of a system, and may thus be a viable rehabilitation option. When considering a particular rehabilitation scheme, existing masonry walls, or their extraction, should be included in the structural analysis along with any new masonry walls that may be added.

A thorough condition assessment of existing masonry walls should be made to increase the level of confidence in characterizing structural properties.

C7.4.1.2 New Masonry Walls

Newly constructed masonry walls can be added to an existing building system for the purpose of strengthening, stiffening, or increasing inelastic deformation and energy dissipation capacity. The design of new masonry walls must follow the *NEHRP Recommended Provisions* (BSSC, 1995). This standard is based on strength design for both unreinforced and reinforced masonry walls. When used in combination with existing walls, no capacity reduction, or ϕ factors, should be used.

In zones of high seismicity, new masonry walls must be reinforced with at least the minimum percentages of reinforcement as specified for a reinforced wall in Section 7.8 (BSSC, 1995). In zones of moderate seismicity, masonry walls must have a minimum of trim bars at corners, top and bottom and around all openings per the *NEHRP Recommended Provisions.* Unreinforced walls can be added to an existing building in zones of low seismicity since they are recognized by this standard.

C7.4.1.3 Enhanced Masonry Walls

Both reinforced and unreinforced walls may be rehabilitated by the various means noted in this section to increase their strength, stiffness, and/or deformation resistance capacity. Enhancement methods are not listed in a priority order, nor are they necessarily the sole methods that can be used.

A. Infilled Openings

A common method of stiffening or strengthening an in-plane masonry wall is to fill window or door openings with masonry. This is typically done for unreinforced walls, but may also be applicable to reinforced walls if needed.

Infilling of an existing opening will stiffen and strengthen a perforated shear wall. The restriction of opening length to no more than 40% of the overall wall length was intended to limit the introduction of new masonry, which by this provision may be considered to exhibit behavior equal to that of the original masonry. The percentage was chosen so that the majority of masonry would be original.

B. Enlarged Openings

Door and window openings in unreinforced masonry walls may be enlarged to alter the aspect ratio of an adjacent pier. By removing a portion of masonry above or below an opening, the height-to-length aspect ratio of the adjacent piers will be increased to such an extent that rocking behavior may govern their response. Although this approach will weaken a perforated masonry wall, it will also increase its inelastic deformation capacity if a ductile rocking mechanism can be invoked. Furthermore, if the method is used, excessive diagonal tensile stresses can be relieved for a relatively stocky pier, thus lowering its vulnerability to nonductile “X” cracking.

The method is also applicable to infill panels. Increasing the size of an opening will reduce infill strength and stiffness and may relieve a surrounding frame from excessive frame-infill interactive forces.

C. Shotcrete

Application of reinforced shotcrete to the surface of a masonry wall is a common method for enhancing both in-plane and out-of-plane strength. The shear area of the wall is increased and the height-to-thickness (h/t) ratio is lowered. Reinforcement embedded in the shotcrete layer substantially improves both the shear and flexural capacities. The method may be used with existing reinforced masonry walls, but has its greatest potential with unreinforced walls.

If shotcrete is used to enhance out-of-plane strength, flexural behavior will be asymmetrical for loading in each direction, since the compression zone will alternate between the shotcrete layer and the masonry.

D. Coatings for URM Walls

Surface coatings may be used to enhance the in-plane shear strength of a URM wall. The h/t ratio will be reduced with the coating, which will enhance the strength of the wall in compression and under transverse loads. Coatings may consist of a cement plaster coating with an embedded steel mesh, or a gypsum plaster coating.

Research has been done on the effectiveness of using fiber-reinforced composites (e.g., kevlar, carbon fibers) for strengthening masonry walls; however, long-term durability remains questionable.

E. Reinforced Cores for URM Walls

Existing URM walls may be reinforced in the vertical direction by grouting reinforcing bars in cores drilled through the wall height. The method, commonly known as the "center core technique," has been used predominantly in California for seismic rehabilitation of URM buildings. With adequate anchorage of new vertical reinforcing bars in the drilled cores, a wall may be assumed to act as a reinforced wall in flexure.

The use of epoxy resins to fill cores around reinforcing bars in older, softer masonry materials has resulted in accelerated deterioration due to incompatibility of materials.

F. Prestressed Cores for URM Walls

Existing URM walls may be prestressed in the vertical direction with strands or rods embedded at their base in grout and placed in cores drilled through the wall height.

Tendons should be ungrouted. Walls enhanced with unbonded tendons will respond in a nonlinear but elastic (returning to undeformed shape) manner. If tendons are bonded with grout, inelastic straining of the tendon can dissipate substantial seismic energy. However, because of the high strength of most tendon steel (cables or bars), excessive compressive strain may result in premature crushing of the masonry before the tendon can develop post-yield strains. Thus, hysteretic damping and ductile performance will be inhibited.

Losses in prestressing force can be estimated based on the expected shortening of a masonry component due to elastic deformations, creep, and shrinkage effects. Design procedures for estimating losses are given in Curtin et al. (1988). Research results on creep and shrinkage movements of clay-unit masonry can be found in Lenczner (1986).

Unlike the reinforced core technique, the prestressed core technique will improve shear strength as well as flexural strength because of the friction that is developed as a result of the increased vertical compressive stress.

G. Grout Injections

The shear strength of existing masonry walls can be enhanced by injecting grout into the interior voids of the wall. For unreinforced brick masonry walls, grout can be injected into possible voids in the collar joint in addition to the head and bed joints. This will also increase the shear and tensile strength between wythes and increase the transverse strength of a multiwythe wall. For hollow-unit masonry, grout can be injected into the open cells.

H. Repointing

Repointing is the process of removing deteriorated mortar joints and replacing with new mortar. Repointing can be used to enhance shear or flexural strength of a URM wall.

I. Braced Masonry Walls

Steel bracing elements can be provided to reduce the span of a masonry wall bending in the out-of-plane direction.

J. Stiffening Elements

Additional structural members can be added to enhance the out-of-plane flexural stiffness and strength of a masonry wall. Such members may be placed in the vertical and/or horizontal direction.

C7.4.2 URM In-Plane Walls and Piers

Walls resisting lateral forces parallel to their plane are termed "in-plane walls."

Solid walls deflect as vertical cantilevered flexural elements from the foundation. Tall slender in-plane walls (height larger than length) resist lateral forces primarily with flexural mechanisms. Squat walls (height less than length) resist lateral forces primarily with shear mechanisms.

Perforated walls can be idealized as a system of piers and spandrel beams. If beams are sufficiently stiff in bending, piers can be assumed to be fully restrained against rotation at their top and bottom. If openings in a perforated wall are relatively large, the wall system will deflect as a cantilevered shear element from the foundation. Pier distortions in flexure and shear will result in story drifts with little rotation of the floor level.

The provisions of Section 7.4.2 apply to both cantilevered shear walls and individual pier elements adjacent to window or door openings. The difference in rotational boundary conditions at the top of either walls or piers is accounted for with an α factor that increases the lever arm of the vertical compressive force about the toe for a pier type component.

C7.4.2.1 Stiffness

A. Linear Elastic Stiffness

Force-deflection behavior of unreinforced masonry shear walls is linear-elastic before net flexural tension stresses at the wall heel exceed tensile strengths, or diagonal tension or bed-joint sliding shear stresses exceed shear strengths.

Laboratory tests of solid shear walls have shown that behavior can be depicted at low force levels using conventional principles of mechanics for homogeneous materials. In such cases, the lateral in-plane stiffness of a solid cantilevered shear wall, κ, can be calculated using Equation C7-1:

$$k = \frac{1}{\dfrac{h_{eff}^3}{3E_m I_g} + \dfrac{h_{eff}}{A_v G_m}} \qquad \text{(C7-1)}$$

where:

h_{eff} = Wall height

A_v = Shear area

I_g = Moment of inertia for the gross section representing uncracked behavior

E_m = Masonry elastic modulus

G_m = Masonry shear modulus

Correspondingly, the lateral in-plane stiffness of a pier between openings with full restraint against rotation at its top and bottom can be calculated using Equation C7-2:

$$k = \frac{1}{\dfrac{h_{eff}^3}{12E_m I_g} + \dfrac{h_{eff}}{A_v G_m}} \qquad \text{(C7-2)}$$

where the variables are the same as for Equation C7-1.

Analytical studies done by Tena-Colunga and Abrams (1992) have shown that linear-elastic models can be used to estimate measured dynamic response of an unreinforced masonry building excited during the 1989 Loma Prieta earthquake.

B. Nonlinear Behavior of URM Walls

As the lateral force is increased on a wall or pier component, flexural or shear cracking—or a combination of both—will occur, resulting in deflections that are nonlinear with respect to the applied forces. Nonlinear behavior of URM walls has been shown to be dependent on the length-to-height (L/h) aspect ratio and the amount of vertical compressive stress.

Behavior of relatively stocky walls (L/h greater than 1.5) is typically governed by diagonal tension or bed-joint sliding, depending on the level of vertical compression, masonry tensile strength, and bed-joint sliding shear strength. For walls governed by diagonal tension, cracks can develop in either a stair-step pattern through mortar head and bed joints, or a straight diagonal path through masonry units. The former action occurs when the mortar is weak relative to the units; the latter occurs when the converse is true. The stair-stepped pattern is better for inelastic deformation capacity because vertical compressive stress normal to the bed joints will result in the development of frictional forces that will remain active at nearly any amount of lateral deflection. Walls governed by a weaker bed-joint sliding shear strength will deform with either a concentrated deformation at one or a few bed joints, or a distribution deformation across several bed joints, depending on the ratio of the cohesion and the frictional coefficient. The inelastic deformability of this sliding type of deformation is also enhanced by frictional forces that remain nearly constant despite the amount of lateral deflection.

In walls with a moderate aspect ratio (L/h between 1.0 and 1.5), considerable strength increases have been observed after flexural cracks form at the heel of a wall as the resultant vertical compressive force migrates towards the compressive toe. As the effective section decreases with progressive cracking, the wall element softens, gradually generating a nonlinear force-deflection relation. If the shear capacity is not reached, the ultimate limit state for such walls is toe crushing. Flexural tension strength at the wall heel does not limit lateral strength. Results from experiments by Epperson and Abrams (1992) and Abrams and Shah (1992) have revealed these tendencies. An analytical study by Xu and Abrams (1992) investigated lateral strength and deflection of cracked unreinforced masonry walls behaving in this range.

For more slender walls (*L/h* less than 1.0) loaded with a relatively light amount of vertical compressive force, flexural cracks will develop along a bed joint near the base of the wall. When the lateral force approaches a value of *PL/2h*, the wall will start to rock about its toe, provided that the shear strength will not be reached. A singularity condition will exist momentarily as the compressive stress at the wall toe increases rapidly just before rocking, which will cause, at worst, some slight cracking at the toe. Despite the fact that a bed-joint crack will develop across almost all of the wall base, the wall can still transfer shear because of friction at the wall toe as a result of the vertical compressive force. After rocking commences, the wall can be displaced to very large drifts with no further damage as a result of the rigid-body rotation about its toe. Again, flexural tension strength at the wall heel does not limit lateral strength. Behavior in this range has been observed with experiments by Calvi et al. (1996) and Costley and Abrams (1995).

The same types of action can be depicted for pier components; however, the vertical compressive force will shift towards the compression toe at both the top and bottom of the pier. This restraining action will cause the rocking strength to almost double because of the increase in lever arm distance between the vertical force couple. The use of the α factor in Equation 7-4, which accounts for differences in rocking strengths for cantilevered walls and fixed-fixed piers, is explained in Kingsley (1995).

Upon unloading, wall or pier components subjected to rocking actions will resume their original position as a result of the restoring nature of the vertical compressive force. For components subject to bed-joint sliding, the slope of the unloading portion of the force-deflection relation will be steep and will continue after the sense of the deflection is reversed. Unlike a reinforced concrete or masonry beam, the hysteresis loop will not be pinched. Thus, the area enclosed by the loop can be large.

C. Lateral Stiffness with Linear Procedures

The linear procedures of Section 3.3 are based on unreduced lateral forces for determination of component actions. If the component is deformation-controlled, these unreduced forces, Q_{UD}, are compared with expected component strengths, Q_{CE}, multiplied by *m* factors representing different ductilities. Because the unreduced forces are fictitious, they cannot be used to assess the expected amount of cracking in any component. Thus, reductions in stiffness cannot be estimated because actual force levels are not known. Therefore, only initial, uncracked linear stiffnesses can be used with the equivalent linear procedures. Any nonlinear action is accounted for by applying the *m* factor to expected strengths.

Much like that of a reinforced concrete beam past yield, the tangent stiffness of a rocking wall or pier is quite small relative to its uncracked stiffness before rocking. For modeling the distribution of story shear to individual piers, the linear stiffness is used rather than the tangent rocking stiffness, which is analogous to the procedure used for strength design of concrete structures. Again, the initial stiffness is used to estimate the elastic demand forces, which are then related to expected strengths by introducing the *m* factor. Thus, individual pier forces are not distributed in accordance with rocking strengths—as is done with FEMA 178 (BSSC, 1992a) or UCBC procedures—but with respect to relative elastic stiffnesses.

C7.4.2.2 Strength Acceptance Criteria

As noted in Section C7.4.2.1B, lateral strength of unreinforced in-plane masonry walls or piers is limited by diagonal tension, bed-joint sliding, toe crushing, or rocking. Net flexural tension stress is not a limit for strength, because post-cracked behavior is assumed for the nonlinear range of response.

Rocking and bed-joint sliding are classified as deformation-controlled actions because lateral deflections of walls and piers can become quite large as strengths remain close to constant. Diagonal tension and toe crushing are classified as force-controlled actions because they occur when a certain stress is reached, and can cause sudden and substantial strength deterioration. Stair-stepped diagonal cracking can also be considered as a deformation-controlled action because frictional forces along bed joints are conserved with vertical compressive forces. However, diagonal tension must be classified as a force-controlled action unless stair-stepped cracking can be distinguished from diagonal cracking through units.

A. Expected Lateral Strength of Walls and Piers

Expected bed-joint sliding shear strength is determined using Equation 7-3. The expected bed-joint shear strength from in-place shear tests is multiplied by the full area of the mortar and/or grout. Although no shear stress can be developed across flexural bed-joint cracks, the increased compressive stress resisted by the opposite wall or pier edge should compensate for this reduction. For the case of a rocking pier, nearly all of the bed joint may be open at the base and top to accept the component's rotation, yet shear is still transferred at the toe because of friction.

Expected rocking strength of walls or piers is determined using Equation 7-4, which was derived by taking moments about the toe of the component. The 0.9 factor accounts for a slight reduction in the lever-arm distance to represent the centroid of the vertical compressive stress. If the component is a cantilevered shear wall, the vertical axial compressive force is assumed to act at the center of the wall at the top, which is the reason for an α term equal to 0.5. If the component is a pier, the vertical force is assumed to act near its edge as the pier rotates and the superstructure remains horizontal. The vertical compressive force, P_{CE}, should be the best estimate of the gravity force during the earthquake.

Lateral strength of newly constructed masonry walls or piers shall follow the *NEHRP Recommended Provisions* (BSSC, 1995).

B. Lower Bound Lateral Strength of Walls and Piers

Lateral strength of walls or piers based on diagonal tension strength is determined using Equation 7-5, which is taken from Turnsek and Sheppard (1980). This equation is only applicable for the range of L/h between 0.67 and 1.00. Because tests do not exist for masonry diagonal tension strength, the bed-joint shear strength, as measured with the in-place shear test, may be substituted where it is assumed that the lower bound diagonal tension strength is equal to the expected value of the bed-joint strength.

Lateral strength limited by toe compression stress is determined using Equation 7-6, which was derived from Abrams (1992). The equation is only applicable for walls or piers loaded with a lateral force that will not result in rocking about their toe. It applies generally to walls with L/h aspect ratios between 1.0 and 1.5 and large vertical compressive stresses. For a lower bound strength, a low estimate of vertical compressive force, P_{CL}, must be used. The limiting compressive stress is conservatively taken as 93% of the lower bound masonry compressive strength, f'_m. Because the lower bound strength is not determined per Section 7.3.2.1, it may be estimated as a fraction of the expected compressive strength, f_{me}.

C. Lower Bound Vertical Compressive Strength of Walls and Piers

The lower bound vertical compressive strength given by Equation 7-7 includes a reduction factor equal to 0.85 to relate prism strength to wall strength, and another factor equal to 0.80 for accidental eccentricities.

C7.4.2.3 Deformation Acceptance Criteria

Unreinforced masonry walls or piers loaded parallel to their plane may experience distress conditions of:

- Minor diagonal-tension or bed-joint cracking
- Major shear cracking or spalling of units
- Loss of strength
- Dislodgment and falling of units
- Out-of-plane movement as a result of excessive rocking

The deformation acceptability criteria given in Section 7.4.2.3 are intended to limit damage accordingly for the goals of each Performance Level.

A. Linear Procedures

For the Linear Static Procedure, *m* factors are given for primary and secondary components for each performance level in Table 7-1.

As discussed in Section C7.4.2.1B., nonlinear force-deflection behavior of unreinforced masonry shear walls has been studied experimentally by a number of researchers. Based on many of these wall tests, and subjective but conservative interpretations of the test data, the *m* factors given in Table 7-1 have been derived. Because the experimental research is by no means sufficiently complete to justify directly every combination of wall aspect ratio and vertical compressive stress, the *m* factors have been calibrated in terms of an approximate value for a square wall panel with a nominal amount of vertical compressive stress. Therefore, for the Life Safety Performance Level, an *m* value equal to 3.5 was established as a control point for development of the table. This value is credible considering that the test data revealed ductilities in excess of five for wall panels with similar characteristics.

Variable *m* factors are given for each Performance Level, corresponding to approximate inelastic deflections associated with specific damage states. For Immediate Occupancy, some cracking can be tolerated for typical occupancy conditions; *m* factors range from 1.0 for bed-joint sliding to 1.5 times the height-to-length aspect ratio for a rocking mechanism. Larger nonlinear displacements can be tolerated for rocking piers because bed-joint cracks in rocking components will close after an earthquake, whereas head-joint cracks resulting from bed-joint sliding will not close fully after the sliding stops. The height-to-length aspect ratio is included in the m factor for rocking piers to relate rigid-body rotation of a component to the lateral deflection at the top of the component. The Life Safety Performance Level is related to lateral deflections associated with the dislodgment of masonry units and/or severe cracking; *m* factors are conservatively set at a value of 3.0 for bed-joint sliding or rocking of square wall or pier components. The Collapse Prevention Performance Level is related to a loss of lateral strength for primary components, and unstable gravity-load behavior of secondary components; therefore, *m* factors are approximately one-third larger than for Life Safety.

B. Nonlinear Procedures

Nonlinear deformation capacities for primary and secondary components are represented in Figure 7-1 with dimensions *d* and *e* respectively. These values are consistent with the *m* values defined for each Performance Level in Table 7-1, and have been extracted from experimental studies on unreinforced masonry walls as discussed in the previous section. The wall drift before strength is lost (the *d* dimension in Figure 7-1) is equal to 0.4% for bed-joint sliding or rocking of square wall or pier components, which is comparable to laboratory test values of approximately 1% for walls that are governed by these deformation-controlled actions. Drift levels have been reduced substantially to 0.10% for walls with zero vertical compressive stress because rocking or bed-joint sliding mechanisms cannot be mobilized, and, as a result, behavior will be governed by force-controlled actions such as diagonal tension.

C7.4.3 URM Out-of-Plane Walls

Walls resisting lateral forces normal to their plane are termed "out-of-plane walls."

C7.4.3.1 Stiffness

Out-of-plane URM walls not subjected to significant vertical compressive stress, and with no restraint at boundaries for formation of arching mechanisms, do not have a nonlinear range. They are brittle elements that will crack under light lateral forces. Depending on the particular Performance Level, cracking of a wall panel may be acceptable if it can be shown that the wall segments rotating about their ends will be stable under dynamic loading.

The stiffness of walls bending about their weak axis is three or more orders of magnitude less than the stiffness of walls bending about their strong axis. Thus, in an analysis of a building system with walls in each direction, the stiffness of the transverse walls will be much less than that of the in-plane walls and can therefore be neglected.

C7.4.3.2 Strength Acceptance Criteria

Out-of-plane walls do not need to be analyzed using the Linear Static Procedure because they act as isolated elements spanning across individual stories. Rather than design on the basis of an equivalent base shear applied to the global structural system (per Equation 3-6 with the Linear Static Procedure), out-of-plane walls should resist inertial forces that are prescribed in Section 2.11.7 without cracking for the Immediate Occupancy Performance Level. For similar reasons, the nonlinear procedures are also not applicable for out-of-plane walls.

The expected demand forces depend on response of the floor or roof diaphragms and the in-plane walls. In addition to the transverse inertial forces resulting from the panel weight, a wall panel must also resist deformations resulting from differential lateral drift across a story, as well as diaphragm deflections. These imposed deflections on the out-of-plane wall panels can be accommodated with cracking of the bed-joints if such cracking is determined to be acceptable for the Performance Level. Even under small amounts of vertical compressive stress, cracked panels will remain stable as they deflect with the attached floor or roof diaphragms.

The out-of-plane response of URM walls may be governed by the development of arching mechanisms in the vertical direction between the floor slabs above and below, or in the horizontal direction between columns, pilasters, or walls running in the normal direction. The type of response mechanism for the out-of-plane wall components is sensitive to the conditions at the panel boundaries and the eccentricities of any applied vertical loads. A rigorous analysis requires knowledge of:

- Accelerations of diaphragms above and below the wall panel
- Edge restraint provided by slabs, beams, or spandrels above and below the wall panel, and by columns, pilasters, or walls at each side of the wall panel
- Masonry compressive strength
- Mortar joint tensile strength
- Eccentricity of vertical compressive loads and amounts of vertical load

In spite of these complexities, the out-of-plane strength of URM walls may be bounded as follows.

- The lower limit of strength is defined for a wall panel with no axial load other than its self weight, no edge confinement from stiff elements above, below, or to the sides, no continuity with adjacent wall panels, and low tensile strength. If such conditions are present, the out-of-plane static strength and stiffness may be considered negligible. However, the panel may be stable under dynamic action for the Life Safety and Collapse Prevention Performance Levels, as the weight of the panel tends to restore lateral response back to its original position.
- The upper limit is defined for a wall panel that is ideally fixed in one or two directions by walls, columns, or pilasters that do not deflect, and vertical compressive forces are applied concentrically about the wall panel. Neglecting masonry tensile strength, flexural cracking will commence when a uniform transverse load, q_{cr}, is applied equal to:

$$q_{cr} = \frac{2Pt}{h^2} \tag{C7-3}$$

where P is the vertical compressive load, and h and t are the panel height and thickness. Because of arching action, the panel can sustain transverse loads with a reasonable upper bound of:

$$q_{cr} = \frac{6Pt}{h^2} \tag{C7-4}$$

At the maximum load level, the wall stiffness can be considered to be negligible; the structural integrity of the panel is dependent on dynamic stability.

C7.4.3.3 Deformation Acceptance Criteria

Acceptance criteria for the Life Safety and Collapse Prevention Performance Levels are based on stable response after cracking of a wall panel has occurred. In addition to the transverse inertial forces resulting from the panel weight, a wall panel must also resist deformations resulting from differential lateral drift across a story, as well as diaphragm deflections. These imposed deflections on the out-of-plane wall panels can be accommodated with cracking of the bed-joints. Even under small amounts of vertical compressive stress, cracked panels will remain stable as they deflect with the attached floor or roof diaphragms. Out-of-plane response of cracked wall panels can be modeled analytically with a dynamic analysis that implicitly considers the motion input at the base of the wall and at the top of the wall. Both the ground motion and the motion of the diaphragm attached to the wall must be determined for this analysis. Research (ABK, 1981) has shown that wall segments should remain stable if their h/t ratio is less than particular values. The values given in Table 7-3, taken from Table C7.4.7.1 of BSSC (1992), are quite conservative relative to the values found in the ABK research. If the h/t ratio of an existing wall exceeds the values given in Table 7-3, and a dynamic stability analysis is not done, then the wall can be either braced (see Section 7.4.1.3I) or thickened with shotcrete (see Section 7.4.1.3C) or a surface coating (see Section 7.4.1.3D). Conversely, the wall may be reinforced (see Section 7.4.1.3E) and analyzed as a reinforced wall, or the wall may be prestressed (see

Section 7.4.1.3F) to increase its cracking moment capacity.

C7.4.4 Reinforced Masonry In-Plane Walls and Piers

This section applies to reinforced wall and pier components that resist lateral force parallel to their plane. Information on modeling lateral stiffness and expected strength of these components is given for flexural, shear, and axial compressive actions.

As for unreinforced masonry wall and pier components (Section 7.4.2), criteria for solid cantilevered shear walls are expressed in the same context as for individual piers between openings in a perforated shear wall.

C7.4.4.1 Stiffness

A. Linear Elastic Stiffness

Before initial cracking, behavior of reinforced wall or pier components is essentially the same as for unreinforced components, because the reinforcing steel is strained at very low levels and the effective area of masonry in tension is usually quite large relative to that of the reinforcing bars. In this range, lateral stiffness of wall or pier components may be determined assuming a linear elastic analysis of components comprising homogeneous materials. Equations C7-1 and C7-2 may be used to determine lateral stiffness of walls and piers, respectively, based on gross uncracked sections and expected elastic moduli of masonry.

For a wall or pier component with sufficient shear strength, flexural cracking will commence at lateral force levels that are a fraction of the ultimate strength. The fraction will depend on the relative amounts of vertical reinforcement and masonry, the reinforcement yield stress, the masonry compressive strength, the length-to-height aspect ratio of the component, and the amount of vertical compressive force. As a result of flexural cracking, the lateral stiffness will reduce, since the masonry is no longer effective in tension. This reduction in stiffness will, however, result in an essentially linear-elastic behavior, provided that the masonry compressive stress remains at approximately one half or less of the ultimate strength and the reinforcement does not yield. Thus, lateral stiffness may be represented with a reduced value representing the effective cracked section.

B. Nonlinear Behavior of Reinforced Masonry Walls and Piers

Reinforced walls are known to soften when cracks initiate. Vertical reinforcement becomes effective after flexural cracks develop along mortar bed joints. With further increase in lateral force, the vertical reinforcement may yield, provided that adequate shear strength is provided. The yielding steel will dissipate substantial seismic energy. In such case, inelastic deflection capacity will be limited by the ultimate compressive strain in the masonry at the wall toe as the steel strains reach well beyond their proportional limit.

Upon unloading, wall stresses will be relieved, but deflections will not reduce substantially because cracks will remain open. When force is reversed in direction, the closing of previously opened cracks will be restrained by the reinforcement acting in compression. In this stage, the resistance of the section is primarily from the reinforcement, and the stiffness will reduce suddenly when the load is reversed. When cracks close fully, the element stiffens, and resumes its character from the loading portion of the previous half cycle. The closing of cracks in the load reversal region causes a “pinching” of the hysteretic loop, which reduces the amount of energy dissipation, and increases the element flexibility. After the first large-amplitude cycle, conventional principles of mechanics used for elements subjected to monotonically increasing loadings cannot be used, because deformations in the masonry and the steel, and at their interface, cannot be estimated reliably. Approximate methods must be used to estimate stiffness and deflection capacity.

Nonlinear behavior of RM wall components has been studied, with large-scale experiments done on: (1) single story walls (Shing et al., 1991), (2) two-story walls (Merryman et al., 1990; Leiva and Klingner, 1991), and (3) a five-story building (Seible et al., 1994). Dynamic testing of reduced-scale, reinforced concrete masonry shear wall buildings by Paulson and Abrams (1990) revealed substantial ductility and inelastic energy dissipation.

C. Lateral Stiffness with Linear Procedures

The stiffness of RM wall and pier components that are cracked can be an order of magnitude less than those components that are uncracked. Because the length of masonry walls in typical buildings can vary, some walls are likely to crack while others remain uncracked. Therefore, lateral stiffnesses should be based on the consideration of whether individual components will crack or not when subjected to expected amounts of vertical and lateral force. This distinction is important when: (1) distributing story shear force to individual walls, or shear force to adjacent piers in a perforated shear wall, (2) estimating nonlinear force-deflection relations for wall or pier components with the Nonlinear Static or Dynamic Procedures, or (3) determining spectral accelerations based on periods of vibration for the Linear Dynamic Procedure.

The following criteria may be used to determine the uncracked or cracked condition states as stated in Section 7.4.4.1.

$$\text{if } Q_{UF} < M_{cr} \text{ then } I = I_g \tag{C7-5}$$

$$\text{if } Q_{UF} \geq M_{cr} \text{ then } I = I_e \tag{C7-6}$$

where:

$$M_{cr} = f_{te} S_g \tag{C7-7}$$

and:

f_{te} = Expected masonry tensile strength per Section 7.3.2.3

I_e = Effective moment of inertia based on cracking

I_g = Moment of inertia based on the uncracked net mortared/grouted section

Q_{UF} = Estimate of the maximum lateral force that can be delivered to the component as defined with Equation 3-15

S_g = Section modulus for the uncracked net mortared/grouted section

The stiffness of a cracked reinforced component can be determined based on a moment-curvature analysis of a particular wall or pier cross section, recognizing the amount and placement of vertical reinforcement, the relative elastic moduli for the masonry and reinforcement, and the expected amounts of axial force and bending moment. Alternatively, the secant stiffness of a cracked reinforced component can be determined using Equation C7-8.

$$\frac{I_e}{I_g} = \left[\frac{15,000}{f_{ye}} + \frac{f_a}{f_{me}}\right]\left[\frac{1}{1 + 0.75(L/h_{eff})^2}\right] \tag{C7-8}$$

where:

f_a = Expected amount of vertical compressive stress based on load combinations given in Equations 3-1 and 3-2

f_{me} = Expected masonry compressive strength as determined per Section 7.3.2.1

f_{ye} = Expected reinforcement yield stress as determined per Section 7.3.2.6

h_{eff} = Height to resultant of lateral force

L = Wall or pier length

Using Equation C7-8, the effective moment of inertia can be determined without considering the amount of lateral force or extent of cracking. This simplification avoids any iterations related to the interaction of demand forces and stiffnesses—a cumbersome process, particularly for deformation-controlled elements where the elastic demand forces, Q_E, are fictitious, as discussed in Section C7.4.2.1C. The derivation for Equation C7-8 can be found in Priestley and Hart (1989).

C7.4.4.2 Strength Acceptance Criteria for Reinforced Masonry

The requirements of Sections 7.4.4.2A, 7.4.4.2B, and 7.4.4.2C are based on the latest revisions to the *NEHRP Recommended Seismic Provisions for New Buildings* (BSSC, 1995) for design of newly constructed reinforced masonry shear walls. The same assumptions, procedures, and requirements are intended for existing wall or pier components.

The lateral strength of RM wall or pier components is governed by either flexural or shear action. The ultimate limit state for flexural action is masonry compressive strain at the wall toe, or tensile fracture of vertical reinforcement. Shear strength is limited by yielding of horizontal shear reinforcement, which causes diagonal tension cracks to widen and, in so doing, reduces aggregate interlock mechanisms. A flexural mechanism should be considered as a deformation-controlled action because it involves yielding of reinforcement and some significant levels of inelastic deformation capacity. Assumptions and procedures for determining expected lateral strength of RM shear walls are given in Section 7.4.4.2A for flexure.

A shear mechanism should be considered as a force-controlled action because it involves diagonal tension of masonry. Assumptions and procedures for determining the lower bound lateral strength of RM shear walls are given in Section 7.4.4.2B.

The resistance of RM walls to vertical compressive stress should be considered as a force-controlled action, and should be characterized by the lower bound strength given in Section 7.4.4.2D.

A. Expected Flexural Strength of Walls and Piers

Expected flexural strength of wall or pier components shall be based on assumptions given in this section, which are similar to those used for strength design of reinforced concrete.

B. Lower Bound Shear Strength of Walls and Piers

Lower bound shear strength of RM wall or pier components is limited to values given by Equations 7-9 and 7-10 for different moment-to-shear ratios. The expected value of masonry compressive strength shall be used to determine these limiting shear forces, which are also considered to be expected values.

Shear resistance is assumed attributable to the strength of both the masonry and reinforcement.

The previous criteria in the *NEHRP Recommended Provisions* (BSSC, 1995) for shear in a plastic hinge zone have been waived, since Equation 7-11 for masonry shear strength is based on tests of shear walls (Shing et al., 1991) where the shear was transferred across a plastic hinge zone. Expected masonry compressive strength, f_{me}, and expected axial compressive force, P_{CE}, are to be used to determine the expected masonry shear strength.

The lower bound shear strength attributable to the horizontal reinforcement is given by Equation 7-12. The previous form of this equation in the *NEHRP Provisions* (BSSC, 1995) has been revised for clarity to the more familiar format used for concrete members. The limit that d_v not exceed the wall height is intended for squat walls (where d_v is larger than h), so that the assumed number of horizontal bars crossing a 45-degree diagonal crack will not exceed the actual number of bars. The 0.5 factor on reinforcement shear strength is taken from research on reinforced masonry shear walls (Shing et al., 1991) and accounts for nonuniform straining of horizontal reinforcement along the component height.

C. Strength Considerations for Flanged Walls

Flanges on masonry shear walls will increase the lateral strength and stiffness appreciably; however, they can only be considered effective when the conditions of this section are met.

The width of flange that may be considered effective in compression or tension is based on research done on reinforced masonry flanged walls (He and Priestley, 1992).

D. Lower Bound Vertical Compressive Strength of Walls and Piers

Equation 7-13 for lower bound axial compressive strength is similar to that for reinforced concrete columns. Lower bound strengths of masonry and reinforcement shall be used, rather than expected strengths. The 0.8 factor represents a minimum eccentricity of the vertical compressive load.

C7.4.4.3 Deformation Acceptance Criteria

A. Linear Procedures

For the Linear Static Procedure, *m* factors are given for primary and secondary components for each Performance Level in Table 7-4. Factors are given to represent variable amounts of inelastic deformation capacity for (1) various ratios of vertical compressive stress to expected masonry compressive strength, (2) wall or pier aspect ratios, and (3) index values representing amounts of reinforcement, expected yield stress of reinforcement, and expected masonry compressive strength.

The *m* factors were determined from an analysis of lateral deflections for reinforced wall or pier elements based on the three parameters included in the table. Curvature ductilities, μ_ϕ, were determined by dividing the ultimate curvature, ϕ_u, by the curvature at first yield, ϕ_y, per Equation C7-9.

$$\mu_\phi = \frac{\phi_u M_y}{\phi_y M_u} \qquad \text{(C7-9)}$$

Displacement ductilities, μ_Δ, were then determined from curvature ductilities, considering plastic rotations at the base of component being limited to a plastic-hinge zone length, l_p, equal to:

$$l_p = 0.2L + 0.04h_{eff} \qquad \text{(C7-10)}$$

which then gave:

$$\mu_\Delta = 1 + 3(\mu_\phi - 1)\left(\frac{l_p}{L}\right)\left(1 - 0.5\frac{l_p}{L}\right) \qquad \text{(C7-11)}$$

Analytical procedures were based on those presented in Paulay and Priestley (1992).

For the Collapse Prevention Performance Level, *m* factors were assigned equal to these displacement ductilities.

Variable *m* factors are given for each Performance Level, corresponding to approximate inelastic deflections associated with specific damage states. For Immediate Occupancy, some cracking can be tolerated for typical occupancy conditions; *m* factors range from 1.0 to 4.0, depending on the amount of vertical compressive stress, the aspect ratio, and the amount of reinforcement. The Life Safety Performance Level is related to lateral deflections associated with the dislodgment of masonry units and/or severe cracking; *m* factors are approximately twice those for Immediate Occupancy. The Collapse Prevention Performance Level is related to a loss of lateral strength for primary components, and unstable gravity-load behavior of secondary components; *m* factors are approximately one-third larger than for Life Safety.

B. Nonlinear Procedures

Nonlinear deformation capacities for primary and secondary components are represented in Figure 7-1 with dimensions *d* and *e*, respectively. These values are consistent with the *m* values defined for each Performance Level in Table 7-4.

Some cracking can be tolerated for Immediate Occupancy. Because of the presence of reinforcement, propagation of cracks will be limited, and thus acceptable wall or pier drifts are larger than those for URM walls.

The Life Safety Performance Level corresponds to severe cracking of the masonry, or a potential for masonry units to dislodge. If spacings of vertical and horizontal reinforcement are equal to or less than 16 inches, these effects will be minimized, and the acceptable drifts contained in Table 7-5 may be increased by 25%.

Severe loss of lateral strength of a wall or pier element can precipitate collapse of a lateral-load or gravity-load structural system. In laboratory experiments, severe loss of strength for in-plane reinforced masonry walls has been observed to occur at lateral drifts exceeding 1.0% for moderate amounts of reinforcement and vertical compressive stress.

C7.4.5 RM Out-of-Plane Walls

Walls resisting lateral forces normal to their plane are termed “out-of-plane walls.” The stiffness of walls bending about their weak axis is three or more orders of magnitude less than the stiffness of walls bending about their strong axis. If a building system contains walls in both directions, the stiffness of the transverse walls will be insignificant. Analysis of out-of-plane walls with the LSP is not warranted, because out-of-plane walls will not attract appreciable lateral forces. Rather than design on the basis of a pseudo lateral load applied to the global structural system (as in Equation 3-6 with the LSP), out-of-plane walls should resist inertial forces that are prescribed in Section 2.11.7. For similar reasons, the NSP is also not applicable for out-of-plane walls. However, the Nonlinear Dynamic Procedure may be useful for out-of-plane walls not complying with strength criteria based on an equivalent static uniform loading.

C7.4.5.1 Stiffness

The static behavior and dynamic response of RM walls bending out-of-plane have revealed very large flexibilities and inelastic deformation capacities. Testing of wall panels is reported by Agbabian et al. (1989), Hamid et al. (1989), and Blondet and Mayes (1991). The effect of flexural cracking on stiffness is quite significant, particularly for small percentages of vertical reinforcement. The stiffness of a cracked section can be as low as one-tenth that of the uncracked section.

C7.4.5.2 Strength Acceptance Criteria

The strength of reinforced out-of-plane walls is nearly always limited by flexural strength, because the span-to-depth ratio is large.

Reinforced masonry walls usually have a single layer of vertical reinforcement that is centered about a single wythe for hollow-unit masonry, or between two wythes of solid masonry. Nominal ultimate flexural capacity can be calculated assuming a rectangular stress block for the masonry in compression, which results in Equation C7-12 for a section with a single layer of tensile reinforcement.

$$Q_{CE} = M_{CE} = A_s f_{ye} d\left(1 - 0.59\rho\frac{f_{ye}}{f_{me}}\right) \qquad \text{(C7-12)}$$

Tests of RM walls have demonstrated the large inelastic deformation capacity of wall panels subjected to out-of-plane loadings. Deformation capacity is dependent on the amount of vertical reinforcement, the level of vertical compressive stress, and the height-to-thickness aspect ratio.

C7.4.5.3 Deformation Acceptance Criteria

Out-of-plane RM walls can resist transverse inertial loadings past the yield limit state with substantial inelastic deformation capacity. If sufficient flexural strength is available to resist the uniform face loading prescribed in Section 2.11.7, and walls are tied to diaphragms at their top and bottom, then they should perform adequately for any level from Immediate Occupancy to Collapse Prevention. Thus, no performance limits are given on out-of-plane deflection of wall panels since post-yield behavior will not need to be relied on.

If the NDP is used, out-of-plane response of the transverse walls may be determined for wall panels performing in the nonlinear range of response. Whereas the out-of-plane walls do not necessarily have to be modeled as part of the global system if strength requirements are met per Section C7.4.3.2, there is no restriction excluding them from a model. On the contrary, inclusion of the out-of-plane walls in a NDP model may be necessary to demonstrate performance for overly slender or weak walls. In such cases, Performance Levels need to be defined in accordance with the estimated out-of-plane deflection of the transverse walls. Because out-of-plane masonry walls are local elements spanning across individual stories or bays, the limit states in the following paragraphs are expressed in terms of lateral deflection across their story height or length between columns or pilasters.

Flexural cracking of an RM wall subjected to out-of-plane bending should occur at the same drift level as for an unreinforced wall. However, this will not, in general, be associated with any Performance Level because cracking of reinforced components is acceptable. As the reinforcement yields at a story drift ratio of approximately 2.0%, cracks will widen substantially and may limit the immediate use of a building.

Life Safety is related to a wall panel reaching its peak strength. This limit state has been estimated to occur at a story drift ratio of 3%, based on experimental research.

The loss of an entire out-of-plane wall may not influence the integrity of the global structural system in the direction under consideration. Therefore, the Collapse Prevention Performance Level should not be applicable for out-of-plane walls. However, the loss of an out-of-plane wall will affect performance of the system in the orthogonal direction when it acts as an in-plane wall. Furthermore, loss of a wall panel can seriously diminish the integrity of the gravity load system if the wall is a bearing wall. Reinforced masonry walls bending out-of-plane are very ductile. Collapse should not occur unless lateral story drift ratios are very large at 5% of the span or larger.

C7.5 Engineering Properties of Masonry Infills

Masonry infill panels are found in most existing steel or concrete frame building systems. Although they are a result of architectural function, infill panels do resist lateral forces with substantial structural action, and should, therefore, be assumed to be part of the primary lateral-force-resisting system.

Since infill panels are usually placed after floors are constructed, they do not resist gravity dead loads at the time of construction. However, if an infill is in tight contact with the beam above, the panel may help support live loads as well as dead loads from upper stories if they are placed after installation of lower-level infills. In addition, if the masonry infill materials tend to expand with time (as is the case with some clay-unit masonry), and/or the frame columns tend to shrink or creep (the case with concrete columns), an infill panel can attract vertical compressive stress as a portion of the gravity loads are redistributed to it from the frame.

In Section 7.5, infill panels are not considered as secondary elements even if they may support gravity loads, because loss of an infill panel should not jeopardize the vertical-load-carrying system. Typically, frames are designed to resist 100% of gravity forces, and should not suffer a loss in structural integrity if the infill panels are eliminated.

If an infill panel is destroyed during seismic shaking, and falls out from the surrounding frame, collapse of the structural system can still be prevented, assuming that the frame resists the full lateral load. If a lateral-force analysis of the bare frame system demonstrates prevention of collapse, then the infill panels should not be subject to limits set forth by the Collapse Prevention Performance Level.

C7.5.1 Types of Masonry Infills

The engineering properties given in Section 7.5 are applicable to building systems with existing, enhanced, or new masonry infills that combine to rehabilitate a building system. In addition, the *Guidelines* provide specific recommendations on minimum requirements for enhancement of existing infill panels, in order that their structural properties may be considered the same as new or existing elements.

Stiffness assumptions, strength criteria, and acceptable deflections for various limit states as described in Sections 7.5.2 through 7.5.3 are common for existing or enhanced masonry infills, or new masonry infills added to an existing building system. Principles of mechanics are the same regardless of the age of a masonry element. Physically, there should be no difference in stiffness assumptions, strength criteria, or inelastic behavior for existing, enhanced, or newly constructed infills. Thus, guidelines on determining engineering properties for each of the three fundamental infill types are expressed in common in these sections.

In Sections 7.5.2 through 7.5.3, infill panels subjected to in-plane lateral forces are separated from walls subjected to out-of-plane forces, because their stiffnesses, strengths, and acceptable deformations are quite different. Unreinforced masonry infills are considered since they are the most common. However, RM infills can be considered with the same criteria, since the in-plane and out-of-plane mechanisms are not influenced negatively by reinforcement.

C7.5.1.1 Existing Masonry Infills

Existing masonry infills will have a significant influence on the lateral strength and drift of a building system. Certain masonry infills may have a brittle character; their removal may improve the overall energy dissipation capabilities of a system, and thus be an acceptable rehabilitation option. When considering a particular rehabilitation scheme, existing masonry infills, or their extraction, should be included in the structural analysis along with any new masonry infill panels that may be added.

A thorough condition assessment should be made of existing masonry infills to increase the level of confidence in characterizing structural properties.

Infilled frame buildings are mostly mid- to high-rise buildings with steel or concrete gravity-load-resisting systems and masonry infill perimeter walls. Steel frame elements are often encased in concrete, brick, or tile for fire protection purposes. For fire protection, masonry infills may also be found within the interior of buildings. Interior infills may extend up to the bottom of beams or slabs, or they may stop at the ceiling level. Floor framing systems in infilled buildings may consist of almost any material. Because infilled frames tend to be significantly stiffer than noninfilled frames, they are likely to be the main lateral-force-resisting elements of the building.

Typical masonry units used for infill panels are clay bricks, concrete blocks, or hollow clay tile. For buildings constructed earlier in this century, masonry units were typically red clay bricks laid in lime mortar. In more recent times, other types of units may have been used, and mortars may have included portland or masonry cement.

Clay-unit infills are common in two or three wythes, and are bonded with headers every five to seven courses. In many cases, the exterior wythe consists of a facing of bricks, decorative terra cotta units, or cast stone (or some combination of these) placed outside the plane of the frame for architectural and weathering purposes. In these cases, the brick wythe is attached to the infill backing with intermittent header bricks or corrugated metal ties placed in the mortar joints. Terra cotta and stone veneers are typically anchored to the infill backing with round metal tie rods bent in the form of staples or hooks.

Location of the infill varies relative to the frame and the connections between infills and frames. Commonly, the interior wythes are supported on top of the beams and the veneer masonry wythe is supported on a steel ledger plate or angle cantilevering out from the beams. In other cases, the outer wythe is supported by the keying action of header bricks interlocked with the interior wythes. The masonry units may be tightly fitted with the surrounding frame units, or gaps may exist between the frame and the infill.

Masonry infills may entirely fill one or more bays and stories in a frame, although this condition is likely only in walls away from the street. More commonly, masonry infills are partial-height infills, or full-height infills with window openings.

Infilled reinforced concrete or steel frames were typically designed to carry all gravity loads and the infills were not intended to be load bearing. Frames were usually not designed for any significant lateral loads. In reinforced concrete frames, beam reinforcement is likely to not be continuous through the joints, and the column bar splices may not be adequate for tension forces. Frame elements may have some widely spaced ties that are not likely to provide adequate shear capacity or ductility.

Steel frames are commonly constructed with rolled shapes for the lighter framing and riveted built-up sections for the heavier framing. Beam connections are usually semi-rigid, with beam seats and clip angles connecting the beam flanges to the column. In some cases, connections with gusset plates may have been used in exterior frames to resist wind loads.

Infilled frames combine nonductile frame systems with brittle masonry materials; hence they conceptually form a poor lateral-load-resisting system. However, observations from the 1906 San Francisco earthquake and other subsequent earthquakes indicate a surprisingly good performance for steel infilled frame buildings. This good performance is attributed to (1) the interaction of the infill with the steel frame, in which the infill provides a significant bracing mechanism for the frame, and (2) the fact that the steel frame members possess adequate ductility to accommodate the demands imposed on them by the infill. In addition, cracking of the infill and the friction between the infill and the frame provides a significant energy dissipation mechanism.

Reinforced concrete infilled frames have not fared as well as steel infilled frames in severe earthquakes, primarily due to the inability of nonductile concrete members to accommodate the demands imposed on them through the interaction with the infill.

Structural frame and masonry infill respond to lateral shaking as a system, both frame and infill participating in the response through a complex interaction. The overall system response and the interaction between frame and infill are influenced by the material and geometric characteristics of each of these elements and the variation of the element characteristics during earthquake response.

The arrangement of infill panels along the height of the building and in plan may have significant influence on the overall earthquake response of the building. This occurs, for example, when framing is kept open at the street side of a building but is infilled along other exterior frames. In this situation, there is the possibility that the resulting asymmetry will produce increased damage due to torsional response of the building. Another case is the lack of infills at a lower story level, which can result in an undesirable soft-story configuration. Similar eccentric or soft-story conditions may be created during the earthquake if infills in a lower story and/or along a side of the building fail, while infill panels in other locations remain relatively undamaged. These overall system concerns can be identified and considered in design if the response behavior of the frame-infill system can be understood and analyzed at the local, single infill panel level.

The failure modes of interest for earthquake performance are as follows.

A. Dislodgment of Masonry Units During an Earthquake

This may result from excessive deformations of the infills due to in-plane or out-of-plane forces, or from inadequate anchorage of veneer courses to the backing courses. Where an exterior wythe of masonry extends beyond the structural frame, delamination or splitting at the collar joint may occur under the action of in-plane loads. Because partial infills and infills with openings are more flexible than solid infills, they may be more prone to this type of damage.

B. Falling of Infill Panels

Infill panels (or large portions of wall) may fall out of the surrounding frame due to inadequate out-of-plane restraint at the frame-infill interface, or due to out-of-plane flexural or shear failure of the infill panel. In undamaged infills, these failures may result from out-of-plane inertial forces, especially for infills at higher story levels and with a large h/t ratio. However, it is more likely for out-of-plane failure to occur after the masonry units become dislodged due to damage from in-plane loading.

C. In-Plane Failure of Infill Panels

Infill panels may lose their strength and stiffness due to in-plane forces imparted to them during earthquake response. This failure mode does not necessarily lead to failure of the overall structural system, although the changes in the strength and stiffnesses of the infill panels are likely to have significant impact on the overall structural response. Also, dislodgment of masonry units or falling of infill panels are likely to follow the failure of the infills due to in-plane deformations. Shear strength of the infilled frame under these circumstances would be expected to be controlled by the shear capacity of the infill. Either of two modes of failure may occur: sliding shear failure along a bed-joint line (commonly about mid-height), or failure in compression of the diagonal strut that forms within the panel.

D. Premature Failure of Frame Elements or Connections

The interaction of the frame with the infill during earthquake shaking results in transfer of interactive forces between frame members and the infill at contact areas. These contact forces may generate internal forces in frame members that are significantly different than those determined by considering lateral response of the frame alone (which has been the usual design assumption in the past). Hence, premature failures may occur in the beams, columns, or connections of the frame. Examples of this behavior are the shear failures induced in columns due to reduced effective flexural length—which may occur when masonry infills form only the spandrels above and below continuous window openings (“captive columns”)—and failures of columns, beams, and connections due to compressive “strut” reactions imparted to them by the masonry infill. Another mode of failure of frame elements is the failure of the tension or compression chords of the infill frame acting as a monolithic flexural element. This mode may predominate in cases where the infill frame is relatively slender and, in particular, where a single bay is infilled in a multibay, multistory building. In this case, the infill frame may act effectively as a flexurally controlled shear wall, with the infill acting as the web and the boundary columns acting as tension and compression chords. Strength in this mode is calculated by conventional flexural procedures, considering the possibility of failure of either the tension chord or the compression chord. Due consideration should be given to tension chord splices, and to tension chord and compression chord offset bars.

E. Failure of the Frame

Upon complete failure of the infill system—provided that no premature failure of the frame elements has occurred—the structural response and performance are determined by the characteristics of the frame only (except, perhaps, for the contribution of the damaged infills to structural damping). As noted above, falling infills present a hazard in themselves, and may also produce a fundamental change in the response of the infill structure. The response of the frame with the infill missing should be assessed, keeping in mind the likelihood that a soft story configuration or stiffness eccentricity may have resulted.

C7.5.1.2 New Masonry Infills

Newly constructed masonry infill panels can be added to an existing building system for the purpose of strengthening, stiffening, or increasing inelastic deformation and energy dissipation capacity.

Design of newly constructed masonry infill panels is not addressed by any existing standards. Procedures for estimating strength and stiffness for new infills shall be in accordance with Sections 7.5.2 and 7.5.3.

C7.5.1.3 Enhanced Masonry Infills

Rehabilitation methods for masonry walls as described in Section 7.4.1.3 are generally applicable as well for masonry infills. In-plane strength and stiffness of a perforated infill panel can be increased by infilling openings with masonry, by applying shotcrete or surface coatings to the face of an infill panel, by injecting grout into the joints, or by repointing mortar joints. Out-of-plane strength can be enhanced with these methods in addition to providing stiffening elements. Enlarging openings is not feasible for an infill panel because panels elements are not susceptible to rocking motions as are masonry piers or walls.

Reinforced or prestressed cores are not practical because vertical coring of an infill panel is difficult.

In addition, the following two enhancement methods are unique to infill rehabilitation.

A. Boundary Restraints for Infill Panels

The stability of isolated infill panels with gaps between them and the surrounding frame may be improved by restraining out-of-plane movements with steel fixtures that are anchored to the adjacent frame members. This method does not fill in the gaps, and therefore does not improve in-plane action.

B. Joints Around Infill Panels

Infill panels with gaps around their perimeter do not fully participate in resisting lateral forces. Furthermore, such walls require perimeter restraints for out-of-plane forces. By filling gaps around an infill panel, multiple benefits can be gained, including increased in-plane strength and stiffness, increased out-of-plane strength (through arching action), and elimination of the need for out-of-plane perimeter restraints.

C7.5.2 In-Plane Masonry Infills

Infill panels resisting lateral forces parallel to their plane are termed "in-plane infills."

Behavior of infilled frame systems subjected to in-plane lateral forces is influenced by mechanical properties of both the frame and infill materials, stress or lateral deformation levels, existence of openings in the infill, and the geometrical proportions of the system. Existence of an initial gap between the frame members and the infill also influences the behavior of the system.

C7.5.2.1 Stiffness

In-plane lateral stiffness of an infilled frame system is not the same as the sum of the frame and infill stiffnesses, because of the interaction of the infill with the surrounding frame. Experiments have shown that under lateral forces, the frame tends to separate from the infill near windward lower and leeward upper corners of the infill panels, causing compressive contact stresses to develop between the frame and the infill at the other diagonally opposite corners. Recognizing this behavior, the stiffness contribution of the infill is represented with an equivalent compression strut connecting windward upper and leeward lower corners of the infilled frame. In such an analytical model, if the thickness and modulus of elasticity of the strut are assumed to be the same as those of the infill, the problem is reduced to determining the effective width of the compression strut. Solidly infilled frames may be modeled with a single compression strut in this fashion.

For global building analysis purposes, the compression struts representing infill stiffness of solid infill panels may be placed concentrically across the diagonals of the frame, effectively forming a concentrically braced frame system (Figure C7-1). In this configuration, however, the forces imposed on columns (and beams) of the frame by the infill are not represented. To account for these effects, compression struts may be placed eccentrically within the frames as shown in Figure C7-2. If the analytical models incorporate eccentrically located compression struts, the results should yield infill effects on columns directly.

Alternatively, global analyses may be performed using concentric braced frame models, and the infill effects on columns (or beams) may be evaluated at a local level by applying the strut loads onto the columns (or beams).

Diagonally concentric equivalent struts may also be used to incorporate infill panel stiffnesses into analytical models for perforated infill panels (e.g., infills with window openings), provided that the equivalent stiffness of the infill is determined using appropriate analysis methods (e.g., finite element analysis) in a consistent fashion with the global analytical model. Analysis of local effects, however, must consider various possible stress fields that can potentially develop within the infill. A possible representation of these stress fields with multiple compression struts, as shown in Figure C7-3, have been proposed by Hamburger (1993). Theoretical work and experimental data for determining multiple strut placement and strut properties, however, are not sufficient to establish reliable guidelines; the use of this approach requires exercise of judgment on a case-by-case basis.

The equivalent strut concept was first proposed by Polyakov (1960). Since then, Holmes (1961, 1963), Stafford Smith (1962, 1966, 1968) Stafford Smith and Carter (1969), Mainstone (1971 and 1974), Mainstone and Weeks (1971), and others have proposed methods and relationships to determine equivalent strut properties. Klingner & Bertero (1976) have found the method developed by Mainstone to provide reasonable approximation to observed behavior of infill panels.

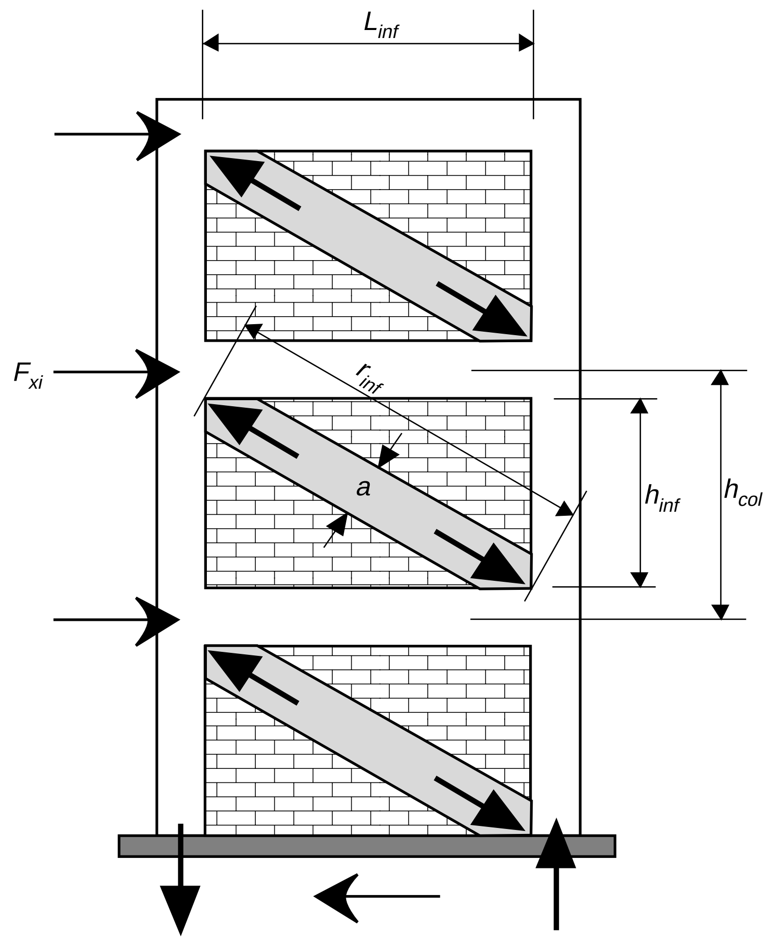


Figure C7-1 Compression Strut Analogy–Concentric Struts

Angel et al. (1994) have found a strut width equal to one-eighth of the diagonal dimension of the infill panel to provide good correlation with experimental results; they also proposed modifications to the frame-infill system stiffness expression developed by Holmes to account for the effects of cyclic loading.

In addition to these empirical studies, frame infill systems have been studied using detailed finite element models (Lotfi and Shing, 1994; Durrani and Luo, 1994; Mehrabi and Shing, 1994; Gergely et al., 1994; Kariotis et al., 1994). Although it is not presently practical to use general-purpose finite element software to perform detailed nonlinear finite element analyses of infill frames, recently developed special-purpose computer software, such as FEM/I (Ewing et al., 1987) may be used to determine equivalent strut properties from nonlinear finite element analyses of typical frame-infill configurations. With such special purpose software, the force-deformation behavior of the frame-infill system is determined through nonlinear finite element analysis, and the equivalent strut properties for use in elastic models are derived from the force-deformation relationship for a target displacement.

Experimental studies done at the Y-12 Plant of the Oak Ridge National Laboratory (Flanagan et al., 1994) showed that the same equivalent strut modeling procedures could be used for infill panels constructed with hollow-clay tile.

In the *Guidelines*, the equivalent compression strut model is adopted to represent the in-plane stiffness of solid masonry infill panels. The relationship used to determine the strut width, Equation 7-14, has been proposed by Mainstone (1971). There are not sufficient data to provide modeling guidelines for representing stiffness of perforated infill wall panels with multiple

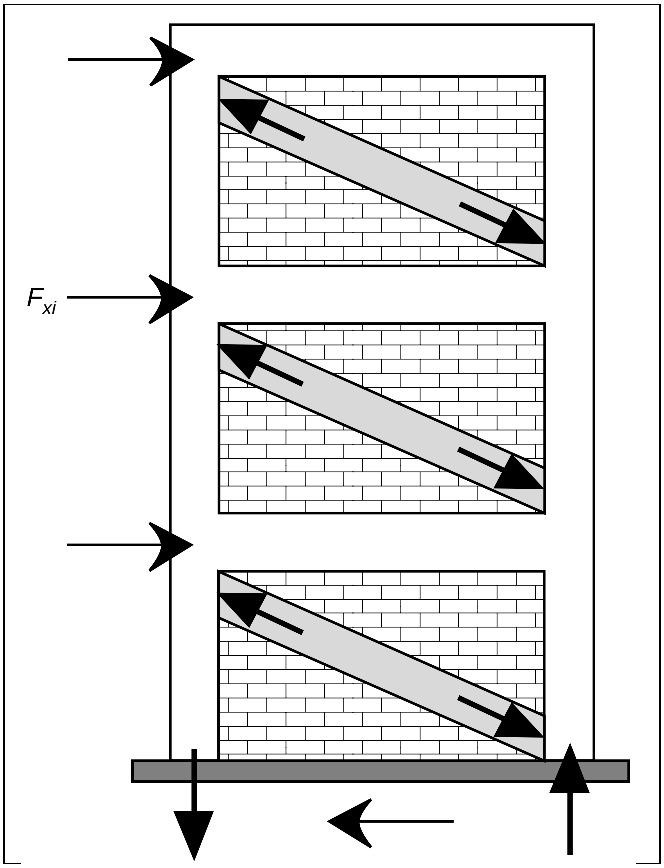


Figure C7-2 Compression Strut Analogy–Eccentric Struts

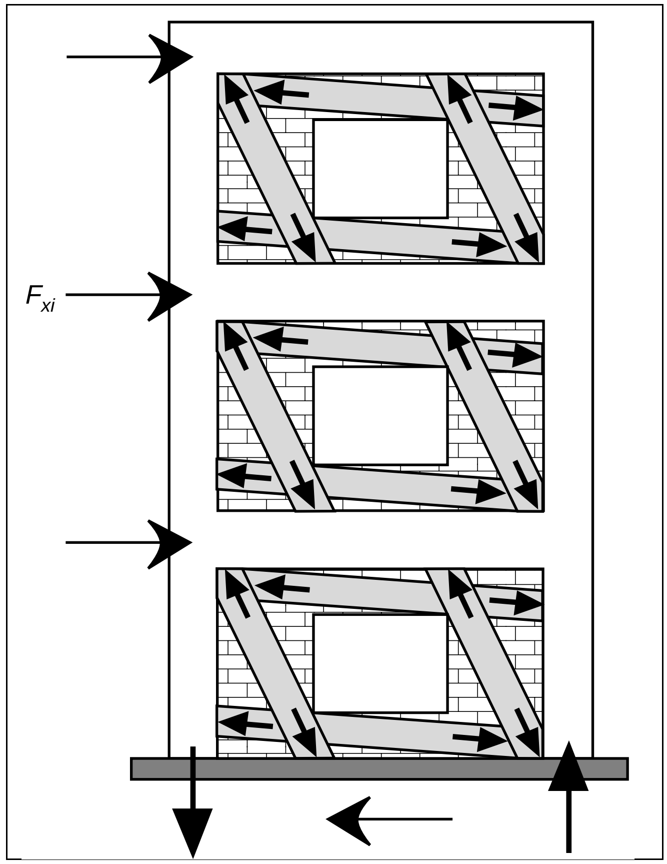


Figure C7-3 Compression Strut Analogy–Perforated Infills

equivalent struts. However, as discussed above, equivalent struts may still be used in analyses of infilled frames with perforated infills, provided that the equivalent strut properties are derived from detailed finite element analyses of representative frame-infill systems.

C7.5.2.2 Strength Acceptance Criteria

A. Infill Shear Strength

The horizontal component of the force resisted by the equivalent strut should be compared with the expected shear strength of an infill panel times the appropriate m and κ factors per the load combination given in Equation 3-18.

The expected infill strength as given with Equation 7-15 is based on an average shear stress across the net mortared/grouted area of a horizontal section cut across the panel. The expected shear strength across this area, f_{vie}, is taken as the expected bed-joint shear strength, v_{me}, for existing construction, or values based on the 1994 *NEHRP Recommended Provisions* (BSSC, 1995) for new construction. No allowance is made for shear strength enhancements due to vertical compressive stress, because gravity forces are assumed to be resisted by the frame.

The expected infill shear strength is based on bed-joint sliding with no confinement from the surrounding frame, and may thus be less than the actual shear strength. A study done by Angel et. al. (1994) found that results from in-place shear tests provide a conservative estimate of infill shear strength. A resolution based on discussions at an NCEER Workshop on Masonry Infills (Abrams, ed., 1994) was that average infill shear stress provided a good index of lateral infill shear strength.

B. Required Strength of Column Members Adjacent to Infill Panels

Infill panels can attract substantial forces to adjacent frame members. These forces can be more demanding of the strength and inelastic deformation capacity of beam and column members than those resulting from lateral design forces applied to a bare frame. Because a stiff masonry infill panel can attract more lateral force than a frame can resist, frames must be checked to see if they are capable of resisting infill forces in the ductile manner that is assumed for their design or evaluation.

Shear strength of the column members should be checked to resist either the horizontal component of the axial force in equivalent struts, or the shear forces resulting from development of plastic hinges at the top and bottom of a column of reduced height. Although neither of these two conditions is exactly representative of what may occur—because of the complex interactions between a frame and an infill panel—these criteria should result in an adequate check to insure ductility of the frame.

The first condition is depicted in Figure C7-4, where the equivalent strut is assumed to be acting eccentrically about the beam-to-column joint with the action illustrated in Figure C7-2. For simplicity, the strut force is assumed to be applied to the column member at the edge of its equivalent width, a. This assumption results in a short shear span of the column equal to l_{ceff}, for which the horizontal strut component must be resisted over. The infill force applied to the frame should be an expected value and not an unreduced elastic demand force as determined with the LSP. The strength of the column member is also an expected strength. Thus, the relative m factors for both the column and the infill panel should be considered when checking the column strength for this action.

Because the first condition can result in excessively high column shear forces, a second option is based on achieving ductile performance of the column when partially braced by the infill panel. This second option consists of checking column shear strength for resisting expected flexural strengths applied at the top and bottom of a short column portion of height l_{ceff}. This requirement may lead to smaller shear forces for relatively light column flexural strengths and will insure that hinging of the column members will occur. The same condition shall be applied to captive columns braced with partial height infills.

Effects of infill panels on frames may be neglected if the bed-joint shear strength of masonry is known to be sufficiently low. In this case, the infill panel will conform to the deflected shape of the frame as courses of masonry slide relative to one another across bed joints. The limit of 50 psi for expected masonry strength defines this sufficiently weak condition, which must be determined from in-place shear tests.

C. Required Strength of Beam Members Adjacent to Infill Panels

For the same reasons as discussed for column members adjacent to infill panels in the preceding section, flexural and shear strengths of beam members must be checked to ensure the transfer of eccentric infill vertical force components. Again, two options are given to check either strength or deformation capacity of the beam. Equations 7-18 and 7-19 are based on the geometry of forces as shown in Figure C7-5.

C7.5.2.3 Deformation Acceptance Criteria

A. Linear Procedures

In Table 7-6, m factors are given only for infill panels acting as primary elements. Because the surrounding frame is assumed to resist gravity forces, the only structural role of the infill is to resist lateral forces, which is a primary action. Thus, infill panels are not considered to act as secondary members and do not need to be checked for their ability to support gravity loads while deflecting laterally.

No m factors are given in Table 7-6 for the Collapse Prevention Performance Level because loss of an entire infill panel should not result in collapse of the frame system. In this case, component behavior is not related to performance of the system. However, the ability of the bare frame to resist gravity and lateral forces must be checked to see if collapse will be prevented.

Amounts of inelastic deformation for an infill panel are expressed in terms of a β factor that expresses the relative frame to infill strength. When the expected lateral frame strength exceeds approximately 1.3 times the expected shear strength of an infill, any sudden loss of infill strength is not likely to result in a substantial decrease in lateral strength of the frame-infill system. Furthermore, when the frame is strong relative to the infill, it will offer more confinement to the infill because inelastic deformations of frame members will be minimized. When the expected strength of the frame is approximately less than 0.7 times the infill expected

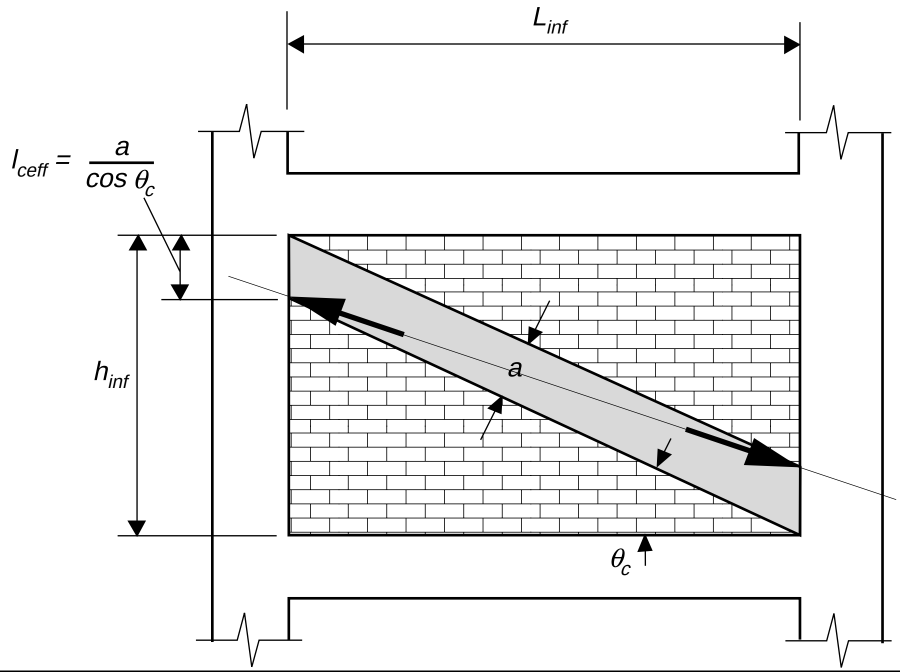


Figure C7-4 Estimating Forces Applied to Columns

strength, a sudden loss of infill strength may result in a sudden and substantial decrease in strength of the frame-infill system. Also, when the frame is weak relative to the infill, confinement effects will be reduced as inelastic deformations of frame members occur.

Inelastic deformation capacity of infills is also expressed in terms of the length-to-height aspect ratio of an infill panel. Larger *m* factors are given for more slender panels than stocky panels because they will be more flexible and thus more adaptable to frame distortions. For taller panels, the angle of the equivalent strut relative to the horizontal will be larger than for stocky panels, and thus offer less resistance to lateral forces.

For the Immediate Occupancy Performance Level, some minor cracking of an infill panel is permissible, and thus *m* factors in Table 7-6 are larger than one inferring that some inelastic deformations can occur. However, when the frame strength is low relative to that of the infill, cracking of the infill can result in damage to the adjacent frame, which could alter the performance of the frame-infill system. Thus, for low β values, the *m* values should be limited to 1.0. For systems with moderate or large β values, no distinction is made in *m* values for the relative frame-to-infill strength because this level of infill should not result in damage to frame members.

B. Nonlinear Procedures

In Table 7-7, inelastic deformation capacities of masonry infill panels are expressed with the *d* dimension, which is given in terms of the generalized force-deflection relations as depicted in Figure 7-1. No values for terms *c* or *e* are given in the table because they apply only to secondary elements. For the reasons discussed in the previous section, infill panels are considered only as primary elements.

Deformation capacity and acceptable deformations are expressed in terms of the relative frame-to-infill strength and the panel aspect ratio, as is done with the *m* factors in Table 7-6.

At a very low level of story drift ratio (on the order of 0.01%), the leeward column of an infilled frame will separate from the infill, resulting in a sudden loss of stiffness. This limit state is of little concern, since the gap will not be visible following the earthquake, and the analysis should have neglected any tension across the gap by using a compression strut. For such a case, the initial stiffness should be based on the axial stiffness of the equivalent strut with properties as defined with

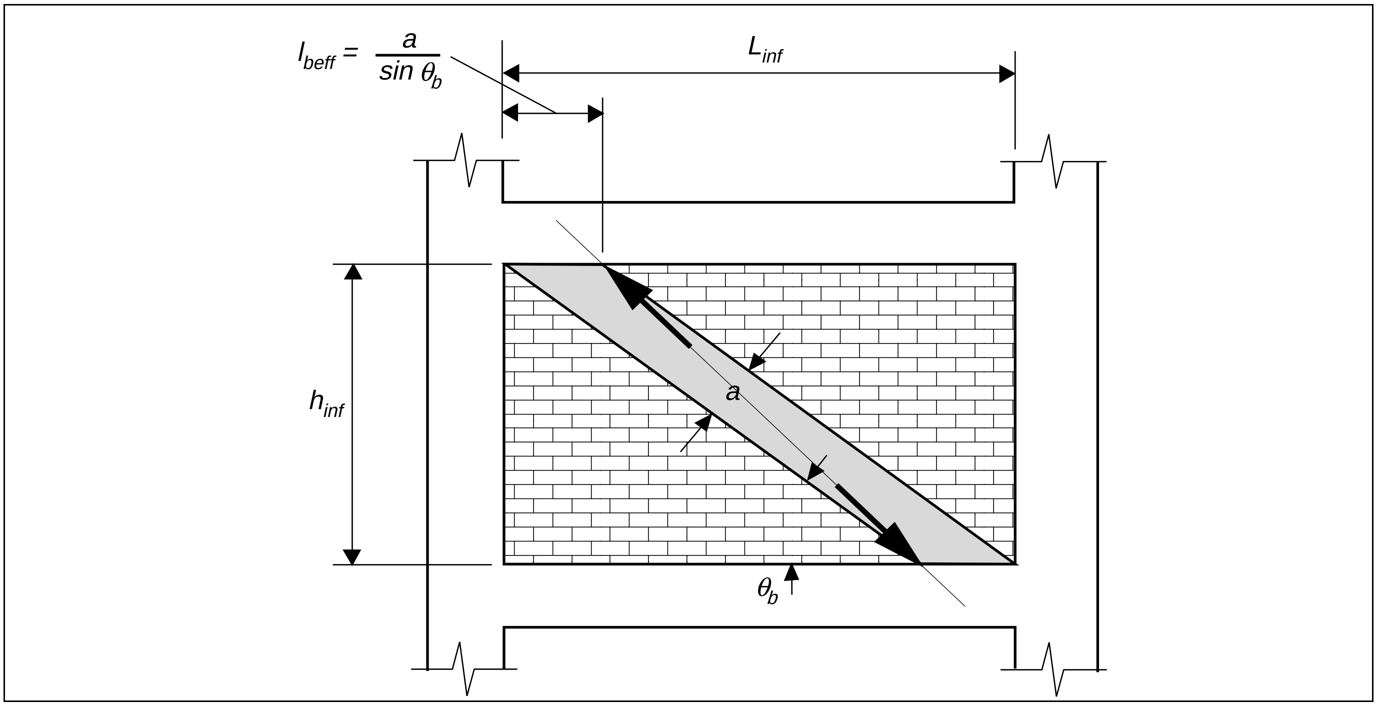


Figure C7-5 Estimating Forces Applied to Beams

Equation 7-14, rather than on a fully uncracked solid panel with full contact with the frame on all edges.

As the infill shear stress is increased, minor cracking along bed joints will develop for weaker mortars, or diagonal cracks will form across a panel for stronger mortars. This will occur at a story drift ratio of nominally 0.1% for square panels. Initial cracking of an infill panel will result in a decreased stiffness, but the panel will still continue to resist increased shear forces if confined by the surrounding frame. Following an earthquake, these minor cracks may be noticeable, but no structural repair would be necessary.

Further loading will result in a wider dispersion of bed joint cracks, or an elongation of diagonal cracks. Moderate or severe cracking of a square masonry infill panel can be expected at story drift ratio levels of approximately 0.3% or more. Even in this condition, an infill panel may continue to provide resistance if the surrounding frame is in tight contact and can provide confinement to the masonry assemblage.

Life Safety corresponds to reaching the peak infill strength. In some cases, Life Safety may also be related to dislodgment and falling of masonry units because of the hazard to life or the blocking of egress. For this case, the relative frame-to-infill strength becomes a significant parameter because post-cracked behavior of a masonry infill panel is very much dependent on the confinement offered by the frame.

Experimental studies done at the Y-12 Plant of the Oak Ridge National Laboratory (Flanagan et al., 1993) showed that the same force-deflection properties could be used for infill panels constructed with hollow-clay tile.

C7.5.3 Out-of-Plane Masonry Infills

Infill panels resisting lateral forces normal to their plane are termed “out-of-plane infills.” The minimum height-to-thickness ratios given in Table 7-8 are based on achieving a transverse infill strength based on an arching action model that will exceed any plausible acceleration level for each of the various seismic zones.

C7.5.3.1 Stiffness

The stiffness of infill panels bending about their weak axes is three or more orders of magnitude less than the stiffness of panels bending about their strong axes. Thus, in an analysis of a building system with infills or walls in each direction, the stiffness of the transverse infills can be neglected.

The out-of-plane deflection of an infill panel can be approximated by considering strips of unit width spanning either vertically between floors or horizontally between columns. The uncracked stiffness of the strip can be considered if the maximum bending moment is less than the cracking moment. Post-cracked behavior can be tolerated, provided that conditions exist for arching action to take place.

The restrictions on when arching action can be considered are based on the ability of the panel to develop internal thrusts when being loaded transversely. The panel must be in tight contact with the surrounding beam and column members. These members must have a flexural stiffness sufficiently high so that they will not flex when subjected to the infill thrust forces, as well as a flexural strength large enough to resist the thrusts.

Slender panels may snap through the frame, particularly if ultimate masonry compressive strains are large at their boundaries. Studies done by Angel et al. (1994) have shown that this may occur for panels with a h_{inf}/t_{inf} ratio exceeding 20 if the ultimate strain is 0.005. This slenderness has been set as a limit on when arching action may be considered.

Transverse deflections at mid-length of a one-way strip for panels that will not snap through the frame can be determined with Equation 7-20, which is a simplified version of an equation given by Abrams et al. (1993) assuming arching action and an ultimate masonry compressive strain equal to 0.004.

C7.5.3.2 Strength Acceptability Criteria

Out-of-plane infills should not be evaluated using the Linear or Nonlinear Static Procedures of Chapter 3 because these infills act as isolated elements spanning across individual stories. The transverse strength of infill panels should exceed the maximum plausible lateral inertial forces that result from the mass of the panel accelerating. Because the evaluation of out-of-plane infill panels does not depend on an unreduced value of base shear—as is done for in-plane components per the LSP—there is no need to use expected values of strength. Thus, strength criteria given in this section are based on lower bound estimates of strength. Actual transverse strengths can be higher.

Masonry infill panels must be restrained perpendicular to the wall surface on all four sides in order to prevent the whole infill panel, or large portions of it, from sliding and falling outward. Exterior wythes of multiwythe infills should be restrained from separating or peeling from the interior wythe (see Section C11.9.1.2A). Field and test observations indicate that infills constructed in tight contact with the surrounding frame can be considered to have adequate out-of-plane restraint. If a gap exists between the frame and the infill on any side, the gap must be filled with grout to provide tight contact, or out-of-plane restraint must be provided with other mechanical means.

Infills that are in tight contact with perimeter frame members develop arching mechanisms when subjected to out-of-plane loads. The out-of-plane capacity of an infill panel can be increased substantially through such an arching mechanism. However, formation of arching mechanisms requires the frame members to have substantial stiffness and strength to resist the thrust forces imparted on them by the arching infill. In general, if the infills are continuous—that is, adjacent bays and story levels are also infilled—the boundary conditions required for arch-mechanism formation may be assumed to be satisfied. For infills with open adjacent bays or story levels, the strength and stiffness of the frame members must be checked to confirm their adequacy.

A lower bound estimate of the transverse infill strength is given by Equation 7-21. The equation is a simplified version of one derived by Angel et al. (1994). Flexibility of beam or column members is included in the expression if their $E_{fe}I_f$ values exceed the minimum of 3.6 x 10^6 lb-in.^{2} as specified in the previous section. According to the theory, frame members with stiffnesses as low as this value should lower transverse strength by as much as 0.6. The lower bound strength equation also includes a reduction of 76% for an estimated amount of in-plane cracking for the most slender panel permitted. In this case, in-plane deflections equal to 50% more than those at initial cracking have been assumed.

C7.5.3.3 Deformation Acceptability Criteria

Because out-of-plane infills are local elements spanning across individual stories and bays, limit states are expressed in terms of lateral deflection across their story height or length between columns.

The Immediate Occupancy Performance Level is not necessarily related to initial cracking of a wall. Some cracking can be tolerated for typical occupancy conditions.

Life Safety is related to extensive cracking of the infill panel. If arching action can be developed, the lateral story drift ratio of the most slender panel permitted (h_{inf}/t_{inf} = 20) according to Equation 7-20 will be 2.8%, which is just less than the limit of 3.0% given for Life Safety. Thus, all infills that can develop arching mechanisms can meet this required Performance Level, provided that their strength will be sufficient to resist inertial forces.

C7.6 Anchorage to Masonry Walls

According to Section 8.3.12 of BSSC (1995), the pullout strength of anchors is governed by the strength of the steel or the anchorage strength of the masonry. When practical, sufficient anchorage should be provided so that the anchor steel will yield, and a brittle pullout failure will be avoided. A ductile anchor will help insure a uniform distribution of force to individual anchors in the case that one or a few anchors are overloaded.

Ductility of an anchor will not significantly influence global ductility of a structural system, because plastic anchor extensions will be quite short relative to inelastic deformations of structural members. Anchors should be considered as force-controlled components, to ensure that the forces delivered to them by adjacent members will be resisted without inelastic straining or pullout of the anchor.

The effective embedment length is the length used to estimate the projected area of a pullout cone of masonry. Per Section 8.3.12 of BSSC (1995), this length is the length of embedment normal to the wall surface to the bearing surface of an anchor plate or head of an anchor bolt, or within one bar diameter from a hooked end.

When the embedment length is less than the minimum length prescribed by Section 8.3.12.1.4 of BSSC (1995), the pullout strength cannot be estimated reliably.

Shear strength of anchorages with edge distances less than 12 bolt diameters can be reduced by linear interpolation to zero at an embedment distance of one inch (25.4 mm).

C7.7 Masonry Foundation Elements

No commentary is provided for this section.

C7.8 Definitions

All definitions for Chapter 7 are given in the *Guidelines*.

C7.9 Symbols

A_v Shear area of wall or pier, in.^{2}

E_{me} Expected elastic modulus of masonry in compression as determined in Section 7.3.2.2, psi

G_{me} Shear modulus of masonry as determined in Section 7.3.2.5, psi

I_e Effective moment of inertia of reinforced wall or pier per Equation C7-8, in.^{4}

I_g Moment of inertia for uncracked, gross section, in.^{4}

I_f Moment of inertia of beam or column member, in.^{4}

L Length of wall or pier, in.

L_{inf} Length of infill panel, in.

M_u Moment at crushing of masonry, lb-in.

M_y Moment at yield of reinforcement, lb-in.

Q_{CE} Lower-bound estimate of the strength of a component or element at the deformation level under consideration

Q_{UD} Deformation-controlled design action

R_1 Out-of-plane infill strength reduction factor to account for in-plane damage

a Width of equivalent strut representing in-plane infill panel, in.

d Effective depth of reinforced section, in.

f_a Expected amount of vertical compressive stress based on load combinations given in Equations 3-1 and 3-2, psi

f_{me} Expected compressive strength of masonry as determined per Section 7.3.2.1, psi

f_{te} Expected masonry tensile strength as determined per Section 7.3.2.3, psi

f_{ye} Expected yield strength of reinforcing steel as determined per Section 7.3.2.6, psi

h_{eff} Height to resultant of lateral force for wall or pier, in.

k Lateral stiffness of shear wall or pier, lb-in.

l_{beff} Assumed distance to infill strut reaction point for beams as shown in Figure C7-5

l_{ceff} Assumed distance to infill strut reaction point for columns as shown in Figure C7-4

l_p Length of plastic hinge for reinforced masonry wall or pier, in.

m Factor to account for inelastic deformation capacity used in Equation 3-18

q_{cr} Uniform transverse load when flexural cracking commences

v_t Wall shear strength, 50th percentile, psi

Δ_{cr} In-plane deflection of infill panel at first cracking, in.

Δ_{inf} Out-of-plane deflection of infill panel at midspan, in.

ε_{mu} Crushing strain of masonry

μ_{Δ} Displacement ductility for reinforced wall or pier section

μ_{ϕ} Curvature ductility for reinforced wall or pier section

θ_b Angle between lower edge of compression strut and beam as shown in Figure C7-5, radians

θ_c Angle between lower edge of compression strut and beam as shown in Figure C7-4, radians

ϕ_y Curvature at initial yield of reinforcement, 1/in.

ϕ_u Curvature at crushing of masonry, 1/in.

C7.10 References

ABK, 1981, *Methodology for Mitigation of Seismic Hazards in Existing Unreinforced Masonry Buildings: Wall Testing, Out-of-Plane,* Topical Report 04.

Abrams, D. P., 1992, “Strength and Behavior of Unreinforced Masonry Elements,” *Proceedings of Tenth World Conference on Earthquake Engineering,* Balkema Publishers, Rotterdam, The Netherlands, pp. 3475–3480.

Abrams, D. P., editor, 1994, *Proceedings of the NCEER Workshop on Seismic Response of Masonry Infills,* Report No. NCEER-94-0004, National Center for Earthquake Engineering Research, State University of New York at Buffalo, New York.

Abrams, D. P., and Matthys, J., 1991, “Present and Future Techniques for Nondestructive Evaluation of Masonry Structures,” *Journal of The Masonry Society*, Boulder, Colorado, Vol. 10, No. 1, pp. 22–30.

Abrams, D. P., and Shah, N., 1992, *Cyclic Load Testing of Unreinforced Masonry Walls*, Advanced Construction Technology Center Report #92-26-10, College of Engineering, University of Illinois at Urbana, Illinois.

Abrams, D. P., Angel, R., and Uzarski, J., 1993, “Transverse Strength of Damaged URM Infills,” *Journal of The Masonry Society*, Boulder, Colorado, Vol. 12, No. 1, pp. 45–52.

Agbabian, M. S., Adham, S., Masri, S., Avanessian, V., and Traina, I., 1989, *Out-of-Plane Dynamic Testing of Concrete Masonry Walls*, TCCMAR Report No. 3.02(b1), University of Southern California, Los Angeles, California.

Angel, R., Abrams, D. P., Shapiro, D., Uzarski, J., and Webster, M., 1994, *Behavior of Reinforced Concrete Frames with Masonry Infills*, Structural Research Series No. 589, UILU-ENG-94-2005, University of Illinois at Urbana, Illinois.

ASTM, latest edition, Standards with the following numbers: C597, C1072, C1197, E518, E519, American Society for Testing Materials, Philadelphia, Pennsylvania.

Atkinson, R. H., Amadei, B. P., Saeb, S., and Sture, S., 1989, “Response of Masonry Bed Joints in Direct Shear,” *Journal of the Structural Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 115, No. 9, pp. 2276–2296.

Blondet, M., and Mayes, R. L., 1991, *The Transverse Response of Clay Masonry Walls Subjected to Strong Motion Earthquakes*, TCCMAR Report No. 3.2(b2), Computech Engineering Services, Inc., Berkeley, California.

BSSC, 1992, *NEHRP Handbook for the Seismic Evaluation of Existing Buildings*, developed by the Building Seismic Safety Council for the Federal Emergency Management Agency (Report No. FEMA 178), 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.

Calvi, G. M., 1988, "Correlation between Ultrasonic and Load Tests on Old Masonry Specimens," *Proceedings of Eighth International Brick/Block Masonry Conference*, Elsevier Applied Science, Essex, England, pp. 1665–1672.

Calvi, G. M., Kingsley, G. R., and Magenes, G., 1996, "Testing of Masonry Structures for Seismic Assessment," *Earthquake Spectra*, Earthquake Engineering Research Institute, Oakland, California, Vol. 12, No. 1, pp. 145–162.

Costley, A. C., and Abrams, D. P., 1995, *Dynamic Response of URM Buildings with Flexible Diaphragms*, Structural Research Series No. 605, University of Illinois at Urbana, Illinois.

Curtin, W. G., Shaw, G., and Beck, J. K., 1988, *Design of Reinforced and Prestressed Masonry,* Thomas Telford Publishers, London, United Kingdom.

Durrani, A. J., and Luo, Y. H., 1994, "Seismic Retrofit of Flat-Slab Buildings with Masonry Infills," *Proceedings of NCEER Workshop on Seismic Response of Masonry Infills*, Technical Report NCEER-94-0004, San Francisco, California, National Center for Earthquake Engineering Research, State University of New York at Buffalo, New York.

Epperson, G. S., and Abrams, D. P., 1989, *Nondestructive Evaluation of Masonry Buildings*, Advanced Construction Technology Center Report No. 89-26-03, College of Engineering, University of Illinois at Urbana, Illinois.

Epperson, G. S., and Abrams, D. P., 1992, "Evaluating Lateral Strength of Existing Unreinforced Brick Masonry Piers in the Laboratory," *Journal of The Masonry Society*, Boulder, Colorado, Vol. 10, No. 2, pp. 86–93.

Ewing, R. D., El-Mustapha, A. M., and Kariotis, J. C., 1987, *FEM/I: A Finite Element Computer Program for the Nonlinear Static Analysis of Reinforced Masonry Building Components*, TCCMAR Report No. 2.2-1, R. D. Ewing and Associates, Palos Verdes, California.

Flanagan, R. D., Bennett, R. M., and Barclay, G. A., 1993, "In-Plane Behavior and Strength of Structural Clay Tile Infilled Frames," *Proceedings of the Sixth North American Masonry Conference*, Drexel University, Philadelphia, Pennsylvania, pp. 371–382.

Flanagan, R. D., Tenbus, M. A., and Bennett, R. M., 1994, "Numerical Modeling of Clay-Tile Infills," *Proceedings of NCEER Workshop on Seismic Response of Masonry Infills*, Technical Report NCEER-94-0004, San Francisco, California, National Center for Earthquake Engineering Research, State University of New York at Buffalo, New York.

Gergely, P., White, R. N., and Mosalam, K. M., 1994, "Evaluation and Modeling of Infilled Frames," *Proceedings of NCEER Workshop on Seismic Response of Masonry Infills*, Technical Report NCEER-94-0004, San Francisco, California, National Center for Earthquake Engineering Research, State University of New York at Buffalo, New York.

Hamburger, R. O., 1993, "Methodology for Seismic Capacity Evaluation of Steel-Frame Buildings with Infill Unreinforced Masonry," *Proceedings of 1993 National Earthquake Conference*, Central U.S. Earthquake Consortium, Memphis, Tennessee, Vol. II, pp. 173–191.

Hamid, A. A., Abboud, B. E., Farah, M. W., Hatem, M. K., and Harris, H. G., 1989, *Response of Reinforced Block Masonry Walls to Out-of-Plane Static Loads*, TCCMAR Report No. 3.2(a), Department of Civil and Environmental Engineering, Drexel University, Philadelphia, Pennsylvania.

He, L., and Priestley, M.J.N., 1992, *Seismic Behavior of Flanged Masonry Shear Walls*, TCCMAR Report No. 4.1-2, Department of Applied Mechanics and Engineering Sciences, University of California at San Diego, California.

Holmes, M., 1961, “Steel Frames with Brickwork and Concrete Infilling,” *Proceedings of Institution of Civil Engineers*, London, United Kingdom, Vol. 19, pp. 473–478.

Holmes, M., 1963, “Combined Loading on Infilled Frames,” *Proceedings of Institution of Civil Engineers*, London, United Kingdom, Vol. 25, pp. 31–38.

ICBO, 1994, *Uniform Code for Building Conservation*, International Conference of Building Officials, Whittier, California.

Kariotis, J., Guh, T. J., Hill, J. A., and Youssef, N.F.G., 1994, “Simulation of the Recorded Response of Unreinforced (URM) Infill Buildings,” *Proceedings of NCEER Workshop on Seismic Response of Masonry Infills*, Technical Report NCEER-94-0004, San Francisco, California, National Center for Earthquake Engineering Research, State University of New York at Buffalo, New York.

Kariotis, J., and Ngheim, D., 1995, “In-Situ Determination of Compressive Stress-Strain Relationship of Multi-Wythe Brick Masonry,” *Proceedings of ASCE Structures Congress*, American Society of Civil Engineers, New York, New York, Vol. 2, pp. 1421-1426.

Kingsley, G. R., Noland, J. L., and Atkinson, R. H., 1987, “Nondestructive Evaluation of Masonry Structures Using Sonic and Ultrasonic Pulse Velocity Techniques,” *Proceedings of Fourth North American Masonry Conference*, The Masonry Society, Boulder, Colorado.

Kingsley, G. R., 1995, “Evaluation and Retrofit of Unreinforced Masonry Buildings,” *Proceedings of the Third National Concrete and Masonry Engineering Conference*, San Francisco, California, pp. 709–728.

Klingner, R. E., editor, 1994, *Performance of Masonry Structures in the Northridge Earthquake of January 17, 1994*, The Masonry Society, Boulder, Colorado.

Klingner, R. E., and Bertero, V. V., 1976, *Infilled Frames in Earthquake Resistant Construction*, Report No. EERC 76-32, Earthquake Engineering Research Center, University of California at Berkeley, California.

Leiva, G., and Klingner, R. E., 1991, *In-Plane Seismic Resistance of Two-Story Concrete Masonry Shear Walls with Openings*, TCCMAR Report No. 3.1(c)-2, Department of Civil Engineering, University of Texas at Austin, Texas.

Lenczner, D., 1986, “Creep and Prestress Losses in Brick Masonry,” *The Structural Engineer,* Institution of Structural Engineers, United Kingdom, Vol. 64B, No. 3, pp. 57–62.

Lotfi, H. R., and Shing, P. B., 1994, “Interface Model Applied to Fracture of Masonry Structures,” *Journal of the Structural Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 120, No. 1.

Mainstone, R. J., and Weeks, G. A., 1971, “The Influence of a Bounding Frame on the Racking Stiffness and Strength of Brick Walls,” *Current Paper CP 3/72*, Building Research Station, Garston, United Kingdom, also published in *Proceedings of Second International Brick Masonry Conference*, pp. 165–171.

Mainstone, R. J., 1971, “On the Stiffnesses and Strengths of Infilled Frames,” *Current Paper CP 2/72*, Building Research Station, Garston, United Kingdom, reprinted from *Proceedings of Institution of Civil Engineers*, 1971 Supplement (iv), Paper 7360 S, pp. 57–90.

Mainstone, R. J., 1974, “Supplementary Note on the Stiffnesses and Strengths of Infilled Frames,” *Current Paper CP 13/74*, Building Research Station, Garston, United Kingdom.

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.

Masonry Standards Joint Committee (MSJC), 1995b, *Specification for Masonry Structures*, ACI 530.1-95/ ASCE 6-95/TMS 602-95, American Concrete Institute, Detroit, Michigan, American Society of Civil Engineers, New York, New York, and The Masonry Society, Boulder, Colorado.

Mehrabi, A. B., and Shing, P. B., 1994, “Performance of Masonry-Infilled R/C Frames Under In-Plane Lateral Loads: Analytical Modeling,” *Proceedings of NCEER Workshop on Seismic Response of Masonry Infills*, San

Francisco, California, Technical Report NCEER-94-0004, National Center for Earthquake Engineering Research, State University of New York at Buffalo, New York.

Merryman, K. M., Leiva, G., Antrobus, N., and Klingner, R. E., 1990, *In-Plane Seismic Resistance of Two-Story Concrete Masonry Coupled Shear Walls*, TCCMAR Report No. 3.1(c)-1, Department of Civil Engineering, University of Texas at Austin, Texas.

NBS, *Evaluation of Structural Properties of Masonry in Existing Buildings*, Building Science Series 62, National Bureau of Standards (now National Institute for Science and Technology), U.S. Department of Commerce, Washington, D.C.

Noland, J. L., Atkinson, R. H., and Kingsley, G. R., 1987, "Nondestructive Methods for Evaluating Masonry Structures," *Proceedings of International Conference on Structural Faults and Repair*, London, United Kingdom.

Paulay, T., and Priestley, M.J.N., 1992, *Seismic Design of Reinforced Concrete and Masonry Buildings*, Wiley Interscience, New York, New York.

Paulson, T. J. and Abrams, D. P., 1990, *Measured Inelastic Response of Reinforced Masonry Building Structures to Earthquake Motions*, TCCMAR Report No. 7.1-1, Structural Research Series No. 555, University of Illinois at Urbana-Champaign, Illinois.

Plummer, H. C. and Blume, J. A., 1953, *Reinforced Brick Masonry and Lateral Force Design*, Structural Clay Products Institute (now Brick Institute of America), Reston, VA.

Polyakov, S. V., 1957, "Masonry in Framed Buildings: An Investigation into the Strength and Stiffness of Masonry Infilling" (English translation), Moscow, Russia.

Polyakov, S. V., 1960, "On the Interaction Between Masonry Filler Walls and Enclosing Frame When Loaded in the Plane of the Wall," *Translations in Earthquake Engineering*, Earthquake Engineering Research Institute, Oakland, California, pp. 36–42.

Priestley, M. J. N., and Hart, G. C., 1989, *Design Recommendations for the Period of Vibration of Masonry Wall Buildings*, Report No. SSRP-89/05, Department of Applied Mechanics and Engineering Sciences, University of California at San Diego, California.

Sansalone, M., and Carino, N., 1988, "Impact-Echo Method, Detecting Honeycombing, the Depth of Surface Opening Cracks, and Ungrouted Tendons," *Concrete International*, American Concrete Institute, Detroit, Michigan, p. 38.

Seible, F., Hegemier, G. A., Priestley, M.J.N., Kingsley, G. R., Kurkchubasche, A. G., Igarashi, A., and Weeks, J. S., 1994, *The US-TCCMAR Full-Scale Five-Story Masonry Research Building Test, Part I - Executive Summary*, TCCMAR Report No. 9.4-1, Department of Applied Mechanics and Engineering Sciences, University of California at San Diego, California.

Shing, P. B., Noland, J. L., Spaeh, H. P., Klamerus, E. W., and Schuller, M. P., 1991, *Response of Single-Story Reinforced Masonry Shear Walls to In-Plane Lateral Loads*, TCCMAR Report No. 3.1(a)-2, Department of Civil, Environmental and Architectural Engineering, University of Colorado at Boulder, Colorado.

Stafford Smith, B., 1962, "Lateral Stiffness of Infilled Frames," *Journal of the Structural Engineering Division*, American Society of Civil Engineers, New York, New York, pp. 183–199.

Stafford Smith, B., 1966, "Behavior of Square Infilled Frames," *Journal of the Structural Engineering Division*, American Society of Civil Engineers, New York, New York, pp. 381–403.

Stafford Smith, B., 1968, "Model Test Results of Vertical and Horizontal Loading of Infilled Frames," *ACI Structural Journal*, American Concrete Institute, Detroit, Michigan, pp. 618–624.

Stafford Smith, B., and Carter, C., 1969, "A Method of Analysis for Infilled Frames," *Proceedings of the Institution of Civil Engineers*, London, United Kingdom, Vol. 44, pp. 31–48.

Tena-Colunga, A., and Abrams, D. P., 1992, *Response of an Unreinforced Masonry Building During the Loma Prieta Earthquake*, Structural Research Series No. 576, University of Illinois at Urbana-Champaign, Illinois.

Turnsek, V., and Sheppard, P., 1980, “The Shear and Flexural Resistance of Masonry Walls,” *Proceedings of the International Research Conference on Earthquake Engineering*, Skopje, Yugoslavia.

Xu, W., and Abrams, D. P., 1992, *Evaluation of Lateral Strength and Deflection for Cracked Unreinforced Masonry Walls, Advanced Construction Technology Center Report #92-26-11*, College of Engineering, University of Illinois at Urbana, Illinois.

C8. Wood and Light Metal Framing (Systematic Rehabilitation)

C8.1 Scope

The scope of Chapter 8 is limited to wood and light metal components and elements that are considered to resist seismic forces as structural members. The chapter includes walls, diaphragms, connections, and other forms of construction. Material is intended to be used with the linear and nonlinear procedures prescribed in Chapter 3. Other wood elements and components are addressed in Chapters 4 and 11.

C8.2 Historical Perspective

C8.2.1 General

The use of wood for building construction is common in most areas of the United States. From colonial times to the present day, many residential structures and smaller commercial, industrial, and institutional buildings have been constructed using wood as the primary building material for the basic structural frame. Generally, the use of wood is limited to small or moderately sized buildings whose superstructures are composed entirely of wood. However, the use of wood as a component or element of virtually every other building type is quite common. Wood floors or roofs in steel frame, masonry, or concrete buildings of extensive size and importance are common in both existing and new construction.

Wood buildings of normal size and shape have performed well in prior moderate earthquakes, with the damage generally limited to nonstructural components. Where large openings, soft stories, and noncontinuous shear walls resulting in offsets of lateral-load-resisting elements exist, as with other materials the performance in significant earthquakes has sometimes been poor. Where torsion of the horizontal diaphragm is utilized to provide seismic resistance, the structures have generally not performed well.

For many years, lateral design of wood buildings typically was based on the assumption that horizontal diaphragms were flexible. Lateral loads were, therefore, distributed to the resisting shear walls based on tributary areas. More recently, it has been recognized that in many cases the relative stiffnesses of the diaphragms and the walls cause the diaphragms to behave more as rigid than as flexible diaphragms. In these cases the loads should be distributed to the walls based on the stiffnesses of the walls rather than the tributary areas to the walls. In addition, it has been general practice to assume that the stiffnesses of the walls were in direct proportion to their length; however, for walls with an aspect ratio greater than one this is generally not true. The effect of bending or overturning can have a greater effect on deflection or wall stiffness than the shear in the wall and distortion of the nails or fasteners.

Due to the relative ease of constructing wood framing, the skill and workmanship of the carpenters and framers should not be assumed. Deviation from codes, accepted plans, and practices is not uncommon. Remodeling and alterations of the structural frame and lateral-force-resisting systems by other trades and building occupants have most likely occurred over the history of the building.

Recently, wood frame construction in urban areas has been extended to three and four stories of apartments or condominiums, often over parking. Many of these buildings, lacking well-conceived designs or good quality construction, have performed poorly in recent earthquakes.

Wood frame residential structures of normal size and shape, even when not specifically engineered to resist seismic loads, generally perform well even in major seismic events. A statistical study of single family detached houses within 10 miles of the epicenter of the Northridge earthquake of January 1994, conducted by the National Home Builders Council, revealed that only a small percentage of the houses performed at levels below the defining characteristics of the Immediate Occupancy Performance Level.

C8.2.2 Building Age

Establishing the age of the building is generally helpful in determining the framing method that may have been used and the materials and structural features that may be found. The age of the building can often be determined from public records and title companies. Local historical societies, preservation groups, city directories, and similar tools—in addition to the architectural features and style of the buildings—can be used to help date the structure.

Buildings constructed prior to 1945 generally will not have plywood sheathing on the floors, roof, or walls. Sheathing of these buildings generally utilized straight or diagonal sheathing boards.

Lumber dimensions have also changed with time. Older structures, built prior to 1940, have members approaching the nominal sizes, while newer buildings have lumber dimensions a half inch to one inch smaller than the nominal size.

Nails have also evolved with time. The early nails were hand wrought. Around 1800, cut nails with a rectangular shank that tapers to a flat point were commonly used. In about 1880 wire nails began replacing the cut nails, but the use of cut nails continued well into the twentieth century. Sampling nails of older buildings can be helpful in establishing approximate ages.

Wood frame walls with wood laths and plaster are commonly found in older wood frame buildings, and were used for interior partition walls of masonry and concrete buildings. Ceilings are often found to be constructed in a similar manner. Wood laths (a quarter inch thick by one and a half inches wide) were nailed to the studs or ceiling joist with one-quarter-inch spaces between the boards. The plaster scratch coat was applied and extruded through the space between the laths, creating physical anchors to the laths. Brown and finish coats were then applied. Over the years, the knobs of plaster have often broken off and the plaster has separated from the wood. Earthquakes often cause sections of the plaster to delaminate from the laths. Plaster ceilings become falling hazards and should be evaluated and corrected as part of the rehabilitation process. For structures constructed prior to 1825, these wood laths are often short, hand split or riven sections of wood that vary in width and thickness. After 1825, the laths were typically manufactured in a mill, resulting in visible circular saw markings. Normally, the laths can be viewed from an attic space by looking at the top side of the ceiling.

Older wood frame buildings were often constructed without plans to show or detail the various conditions and connections of the elements. The standard practice and skill of the carpenter were relied upon to obtain adequate connections. Generally, these older buildings were not systematically designed for the effects of lateral loads, but utilized conventional construction, and were “deemed to comply” with requirements. Many small wood frame buildings in most areas of the country continue to be designed and built on this basis. Load paths tend to be random, with critical ties or connections often completely overlooked.

C8.2.3 Evolution of Framing Methods

Post and beam, half timber, and frame construction are 18th and early 19th century techniques in which posts and beams were used as the general framing method, with the posts at the exterior walls placed three to five feet on center and extending the full height of the building. The spaces between the posts were filled with masonry of various types. This method, although extensively used throughout western Europe and the British Isles, was not generally successful in New England; harsh winters led to the deterioration of the masonry fill materials. Wood siding or brick veneers were found to be more appropriate for the climate. Diagonal bridging or braces between the posts provide the lateral bracing for the walls; these diagonal and vertical members are often exposed on the exterior surface. Many modern frame structures attempt to duplicate the architectural appearance by using exposed boards on or between exterior plaster or brick veneer; however, this is strictly architectural and does not contribute to the lateral strength of the building.

The advent of balloon framing in the early 19th century made the frame building construction techniques essentially obsolete. Appearing around 1830, this new lighter framing method was devised using 2" x 4" and 2" x 6" studs spaced at 16 or 24 inches on centers. The term “balloon framing” arose because the system appeared to be so light when compared with the post and beam or frame system. Balloon framing replaced the post and beam or frame method in the Midwest by 1840; however, it did not spread to the east and west coasts until the 1860 to 1870 time frame. In balloon framing, the studs generally ran the full height of the structure from the first floor to the roof. For multistory structures, the floor framing was supported by a let-in ribbon and the joists were nailed into the sides of the studs. Lateral bracing was achieved by the inclusion of diagonal blocking between the studs, by braces let-in to the studs, or by the finish materials for the interior and exterior walls. Horizontal diaphragms generally consisted of either straight or diagonal sheathing boards.

The balloon framing method creates a poor connection condition for seismic resistance between the floor diaphragm and the exterior wall, since the diaphragm stops at the interior face of the wall and no shear connection is generally present. Around 1910, balloon framing was rapidly replaced by the development of western or platform framing; however, the term balloon framing is still used to indicate full-height studs at a gable or sloped roof condition where no intermediate top plates are present. The major difference between platform framing and balloon framing was that each level of the structure was now constructed separately. The wall framing members are the same as those used for the balloon method, and unless the floor to wall connection is exposed, it is virtually impossible to tell the difference in the building types. Platform framing is the method currently employed for multistory wood frame construction. In the earlier platform framing buildings, bracing was obtained in a fashion similar to that for balloon framing, but in contrast to the balloon frame method, the floor sheathing diaphragm extends out to the exterior wall, resulting in a more positive connection between the exterior walls and floor diaphragm for shear transfer.

For both balloon and platform framed buildings, the finish materials on the stud walls usually provide the lateral resistance for the structure. These materials often perform in a brittle fashion and undergo extensive cracking. Wood lath and plaster wall finish continued to be employed through the 1940s, when they were replaced with gypsum lath or button board and plaster. However, in the mid- to late 1960s gypsum wallboard or drywall—which had been developed some 30 years earlier—became popular, and is now the general finish material in use for interior walls and partitions for both residential and commercial construction.

With the evolution of structural panels, plywood and oriented strand board are typically utilized for both horizontal and vertical lateral bracing systems.

Single side wall construction is a unique type of construction generally used only for barns, out-buildings, and cottages in rural and semi-rural areas. The construction utilizes one-inch vertical boards for the exterior walls, with a sill plate at the base and a top plate for connection of the boards. Spaces between the boards (usually 1" x 10"s or 1" x 12"s) are covered by vertical battens, generally 1" x 2"s. These are generally very low-mass structures, and seismic loads are not usually the critical loading criterion for lateral design. In some residential buildings, single side wall construction has been utilized for interior walls in a similar fashion by using one-inch tongue and groove wood boards vertically. This type of construction is no longer permitted by codes.

The development of three- and four-story multifamily structures created a new set of problems relating to the stacking of tall, narrow shear panels, generally at exterior walls. These shear panels are so flexible that they are often ineffective for resisting loads without large associated deflections. These deflections can result in extensive damage to finish materials and the distribution of loads to walls or components not intended in the design to act as part of the lateral-force-resisting system, or to carry the magnitude of load imposed.

The need to provide for parking at the ground level of buildings often creates seismic resistance problems. As is the case for all construction materials, the interruption of the upper level shear walls at the lower levels, where a garage requires large openings, creates soft story effects or, in some cases, torsional effects that may result in deflections in the support frame at the parking level beyond the limited capacity of the frame to maintain lateral stability.

Prior to the common usage of concrete slab-on-grade construction for residential, commercial, and institutional wood framed buildings, the buildings were typically constructed on raised foundations, sometimes incorporating short wood stud walls below the first level, called cripple walls. This results in the lateral loads from interior walls transferring to the exterior walls, placing an extra demand on the wall-to-foundation connection and the cripple walls. These cripple walls have performed poorly in past earthquakes and generally need to be enhanced by the addition of structural panels.

Light gage metal stud walls, floors, and roof joists have been used for the construction of small structures, sometimes in combination with wood members. The members are generally formed into channel or “C” shapes. Each fabricator varies the size and shape somewhat in order to accommodate various features such as nesting or splicing of sections and, in some cases, the ability to apply finish material with nails. Some shapes have webs punched out in various patterns to allow the passage of conduits or the inclusion of bridging between the studs.

C8.3 Material Properties and Condition Assesment

C8.3.1 General

Before an analysis of an existing building can be conducted or an attempt to strengthen or upgrade the structure can be made, the features of the existing structure must first be determined. The lateral-force-resisting system must be identified and the various elements located and evaluated.

The evaluation process can be conducted at several levels of effort, from a simple walk-through to a complete removal of finish surfaces, along with sampling and testing of existing materials or a mock-up test of existing assemblies. For most buildings, the evaluation of existing conditions will involve the removal of some finish materials so that the structural elements and their condition can be inspected and established.

The analysis should reveal those elements that are critical to the performance of the building. Where high load to capacity is indicated, more effort and intensive inspection of the existing condition and elements should be done.

Mechanical properties of wood are affected by moisture, temperature, load history, and presence of decay. Existing in-place properties may vary significantly from those specified on design drawings or those prevalent in the building's era of construction.

Personnel involved in the quantification of material properties shall be highly experienced in testing practices, proper use and application of methods and procedures, and interpretation of results.

In general, existing wood components that have been subjected to a relatively dry environment (e.g., interior or protected exterior location) and normal loading history will likely possess near-original mechanical properties. However, components exposed to the weather or to an unusual loading history, such as heavy static or dynamic loading, may have reduced mechanical properties. The design professional must also consider these factors when establishing properties and the testing/condition survey protocol.

The performance of wood buildings subjected to seismic loading is, to a great extent, dependent on the connections of the various elements in order for the various parts of the building to remain connected under loads or distortions beyond the “elastic” range.

C8.3.2 Properties of In-Place Materials and Components

C8.3.2.1 Material Properties

Generally, the type of wood used in a particular geographic area is dependent on the availability of the various species at the time of construction. Higher grade lumber is often found in older buildings. If the wood is not easily identified visually, core samples can be taken for identification by experts in wood science.

The grade of the existing material will have to be determined by inspection. However, the condition assessment of the various elements and the existence of proper connections are more important to the performance of the structure than the grade of the material used in the structure.

Where existing framing is covered with finish material, attic spaces and underfloor crawl spaces can be used for a preliminary evaluation to view the type and grade of framing without having to damage or remove finishes.

In some cases, inspection may reveal members or elements that have been heavily damaged by insects or decay. These members will have to be replaced regardless of the load or stress level present. Cores can be taken vertically through glue-laminated beams to evaluate the adhesives used and to test the shear capacity between the laminations.

No matter which method of analysis is used in the rehabilitation effort, a continuous load path is required between the foundation and the walls, frames, floors, and roof of the structure. A missing or weak link between elements in the system will have a serious effect of the performance of the building as a whole.

For performance above the Life Safety Performance Level, the traditional method of design and analysis—assuming that wood diaphragms are flexible and that loads are distributed to resisting elements on a tributary area basis, or that the loads in the various walls are in proportion to their length—is not appropriate. The relative stiffness, including bending and overturning effect of walls, must be considered, and the deflections of the various element must be calculated, rather than relying on arbitrary aspect ratios in order to limit anticipated distortions.

For all steel stud systems with diagonal straps or rods for lateral bracing, the provisions of Chapter 5 should be used. For systems using wood panels for bracing, see Section 8.4 for analysis and acceptability criteria.

C8.3.2.2 Component Properties

A. Elements

Refer to Section 8.4 for a description of the various types of shear walls that might be found in an existing building, and Section 8.5 for a description of the horizontal diaphragms. For existing shear walls, it is recommended that some walls be exposed and the nails and conditions examined for proper construction. Nails smaller than specified, overdriven nails, and ineffective nails lacking proper edge distance can significantly reduce the capacity of the walls or horizontal diaphragms.

Components of the lateral-force-resisting system are most likely to be absent or deficient in all but the most recently built existing buildings. These elements are all required for the full development of the load path necessary to deliver the various loads and forces to the resisting elements. Where they are missing, or inadequately designed or constructed, the structure is likely to undergo damage and distortions that could result in local failures or, in some cases, extensive damage to the entire structure. Dramatic catastrophic failures in prior earthquakes have brought about the requirements for some of these components—such as the need for crossties to extend across buildings in order to anchor heavy wall elements and the need to provide ties or collectors at inside corners or wall offsets to carry loads into the walls at those locations. The presence or absence of chords on a diaphragm has a dramatic effect on the magnitude of deflection that the diaphragm will experience when subjected to lateral loads (see Section 8.5).

Nominal and standard dressed size cross-section dimensions are published in the Supplement to the *National Design Specification for Wood Construction* published by the American Forest & Paper Association (AF&PA, 1991), or in publications by the American Institute of Timber Construction (AITC), American Plywood Association (APA), and other organizations. The era of original construction also dictates sectional dimensions (e.g., size of 2" x 4" studs). Variance in these dimensions is also small, and their effect should not affect component strength or deformation calculations unless they are attributed to a degradation process, excessive shrinkage, or creep.

B. Connections

As with all construction materials, and as stated in the *Guidelines*, connection methods are critical to building performance. The type and character of the connections must be determined by a review of the plans and a field verification of the conditions. Connection capacity limits the magnitude of load that can be delivered to a connected element; connections should be upgraded to the extent that the connected element can resist the load.

The *m* values given in Table 8-1 for evaluating the connections are based on recent research on wood connections at Virginia Polytechnic (Dolan et al., 1994) on cyclic behavior of nails and bolts. Values for screws and lag bolts were estimates based on perceived performance. Past tests of cyclic performance of shear walls—with screws in lieu of nails—have indicated a lack of ductility. The threads on the screws appear to cause a stress concentration that results in a brittle type failure of the screw with a low number of cycles of load.

When evaluating bolted connections, a large amount of the movement that occurs in the connection is due to the oversize condition of the holes for the bolts, in both the wood and the steel, where applicable. Poor workmanship can result in excessive movement in the joints. Removal and inspection of some bolts in deflection-critical joints will give an indication as to the quality of the work and the amount of movement to be anticipated. When adding bolts to existing connections, it is recommended to match the existing bolt sizes. It should also be noted that smaller bolts have been shown to have more ductility than larger bolts.

Connections of heavy concrete or masonry walls to wood roofs or floors have been shown to be a problem in prior earthquakes. Even where positive metal strap ties are present between the wall and wood framing member, failures have occurred at bolt hole locations due to a lack of ductility in the anchor strap.

C8.3.2.3 Test Methods to Quantify Properties

Certain field tests—such as determination of wood gradation and moisture, and estimation of stress level—

may be performed, but laboratory testing on clear, straight-grained samples removed from existing construction must be done if confirmation of field tests is desired. Particular laboratory test methods that may be employed include measurement of moisture content and specific gravity, direct tensile and compressive strength, preservative presence, and connector strength and withdrawal resistance, as well as other mechanical property tests. For each test, industry standards published by the American Society for Testing and Materials (e.g., ASTM D143, D196, D1761, D1860, D2555, D2915, F606) shall be followed.

Quantifying material properties for most connection components, including bolts and nails, is relatively simple. The individual components can be visually inspected and removed (without disturbing physical condition) for evaluation in the laboratory. Expected strength properties for these connectors may be derived from standard laboratory tests similar to those provided in Section 5.3 for steel components and connectors. The influence of connector material properties on behavior of pinned and simple shear connections is generally well understood. However, the multitude of possible configurations and orientations of the connectors may complicate connection analysis. When removing connectors, the condition of the installation shall be noted. Oversized holes or splits in the wood at the connection will prevent the element from full participation.

For structures with archaic or nontraditional wall bracing systems, and where the performance is unknown and it is desired to use the existing elements, a mock-up cyclic test can be conducted to determine the envelope of the hysteretic behavior. From this data, the appropriate *m* factors and control points on the idealized nonlinear distortion backbone curve can be determined.

C8.3.2.4 Minimum Number of Tests

For all laboratory test results, the mean yield and ultimate strength may be interpreted as the default strength for component strength calculations if the coefficient of variation in results is less than 20%. For results with higher variation, to 30%, the expected strength shall be taken as the mean value less the average coefficient of variation as derived via simple statistics. If variabilities higher than 30% are witnessed, further testing shall be performed to identify the source. Such testing shall involve increased sampling and testing in all primary components at each floor level. This result may also indicate the presence of differing material grades in the structural system. Use of ultimate strength values in component capacity calculations shall be based on industry-accepted practices.

If a higher degree of confidence in expected strength values is desired, the sample size shall be determined using ASTM Standard E 22 guidelines. Alternatively, the prior knowledge of material grades from Section C8.3.2.5 may be used, in conjunction with Bayesian statistics, to gain greater confidence with the reduced sample sizes and test results noted above.

C8.3.2.5 Default Properties

The traditional method for designing wood frame buildings and the wood members and elements of other types of buildings has been the allowable stress method. All of the code and material reference standards provide information based on the allowable stresses of the members. The in-grade testing program conducted by AF&PA determined that the limit state or ultimate strength of the materials was, on average, 2.16 times the allowable strength. The load duration factor recommended in the more recent codes for seismic loading is 1.6. A yield load of 80% of ultimate gives a combined factor of 2.76. Therefore, use of a factor of 2.8 is recommended, until such time as the codes and standards are revised to provide the limit state values as appropriate. Other capacity reduction factors—such as moisture exposure, and presence or absence of checks or cracks—should be included in the capacity determination.

The deformation values for the various connectors are based on the cyclic tests of nailed and bolted connections of various types, which were conducted by Dolan et al. (1994). Screw and lag bolt values were estimated from the test data.

C8.3.3 Condition Assessment

C8.3.3.1 General

The features of the existing structure must first be determined. This can be based on field measurements of the building or, ideally, from a set of record construction documents. With many existing structures, especially smaller wood frame structures, plans are not available. Searches of current and former owners', architects', engineers', and city or county records, and contractor files can sometimes yield valuable information concerning an existing structure; these resources should be investigated.

An estimate of the mass of the structure is required in order to determine the seismic load demand on the structure, irrespective of the analysis method used (and even if the Simplified Rehabilitation Method is used). Thus, the size and condition of all the various parts of the building must be determined in order to establish the dead load of the building.

A predetermined systematic methodology needs to be established to determine the character of the lateral-force-resisting elements and the specific connections or load transfer elements that are to be investigated. The investigation should include critical locations in the building as well as a general condition survey. A preliminary analysis will determine these critical element locations or "hot spots" so that the expense and inconvenience of removing otherwise serviceable finish surfaces can be controlled and limited.

After the preliminary analysis has been completed, a more detailed investigation of the building can be conducted on those elements and connections that are critical to the building performance with a high load demand to capacity ratio (DCR).

C8.3.3.2 Scope and Procedures

All of the primary lateral-load-resisting elements of the structure need to be assessed as to their features and conditions. This will often involve the removal of finish materials to observe the existing conditions. The availability or absence of record drawings has a great effect on the amount of removal required.

The following paragraphs identify those nondestructive methods having the greatest use and applicability to assessment.

- Surface Nondestructive examination (NDE) methods for wood components include coring, drilling, probing, and sounding. These methods may be used in parallel with visual inspection to find surface degradation such as decay, splitting, service-induced cracks, and other degradation. These methods do not require significant equipment, but depend on suitable access and expertise in application for successful results. Moisture meters may also be used to assess the presence of decay and conditions producing reduced mechanical properties.

- Volumetric NDE methods, including radiography and ultrasonic stress wave testing, may be used to identify the presence of internal discontinuities in base materials, as well as to identify loss of section or strength. Ultrasonics is particularly useful because of the ease of implementation and the ability to estimate elastic properties of the wood (if density is known). Volumetric NDE of wood requires significant expertise because of the number of variables that may influence results.

- 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 component and element strength.

- Reinforcing location devices can be used to verify the presence of metal hardware at various locations. Some of the locations will still need to be exposed to verify the electronic results and to determine the number of nails, bolts, and other hardware.

C8.3.3.3 Quantifying Results

As previously noted, in the absence of degradation, component section properties have been found to be statistically close to nominal published values. Unless splitting or other mechanism is observed in the condition assessment as causing sectional loss, the cross-sectional area and other sectional properties shall be taken as in the design drawings. If some material damage has occurred, the loss of wood or connector capacity shall be quantified via sampling and laboratory testing. The sectional properties shall then be reduced accordingly, using the laws of structural mechanics. If the degradation is significant, rehabilitative measures shall be undertaken on the deficient component(s). The connection of the members and elements warrants special attention, as failures often occur at the connection rather than in the members or elements themselves. Existing condition may result in both a reduction in capacity and a reduction in ductility, which must be evaluated and incorporated into the analysis.

C8.3.4 Knowledge (κ) Factor

The assignment of knowledge (κ) factors is to a large extent dependent on the availability of a reliable set of plans for the original building. Older building plans for wood frame structures often contained very little structural information and are thus of minimal use; in such cases, the structure should be classed along with those for which no original plans are available. Where new elements are being installed, the κ factor is not applicable. New elements should be designed in accordance with the *NEHRP Recommended Provisions for New Buildings* (BSSC, 1995), incorporating the appropriate phi (ϕ) factor where applicable.

Using the defined κ factor and allowable stresses derived from testing or other source (e.g., *National Design Specifications*, AF&PA, 1991), the stress capacity of the component(s) for different limit states may be established. Adjustment of the stress capacity may be applied on a composite basis to the building or on the basis of individual components. For components with strengths derived from testing, the capacity and deformation limits shall be adjusted by multiplying the strength or deformation limit by a κ factor of 1.0. For wood components not tested and found in fair or better condition, with limited amounts of warping, splitting, or other minor degradation, the capacity shall incorporate a κ factor of 0.75. For wood found in poor condition, rehabilitative measures shall be undertaken, with attention paid to mitigating the cause for existing degradation. In all capacity calculations, the adjustment factors for size, environmental conditions, and load history shall be considered. For connections where plans do not exist, the condition must be exposed to establish the number and size of bolts, nails, and other connections. These exposed connections should utilize a κ factor of 1.0. If all connections are not exposed, but assumed to be similar to those exposed, a κ factor of 0.75 should be used.

C8.3.5 Rehabilitation Issues

Structural panels are used to provide lateral strength and stiffness to most modern wood frame buildings, and are generally recommended for the retrofit or rehabilitation of horizontal diaphragms and shear walls of existing buildings. The system relies on the in-plane strength and stiffness of the panels and their connection to the framing. Panels are connected together by nailing into the same structural member to, in effect, create one continuous panel. The various panels listed have different strengths and stiffnesses; they are discussed and described in Sections 8.4 and 8.5. The performance of the structural panels is dependent to a great extent on the nailing or attachment to the framing. The nail spacing and effectiveness of the attachment should be investigated if the existing panels are to be relied upon to withstand significant loads. If nails are to be added to existing panels they should be of the same size as the existing nails.

C8.4 Wood and Light Frame Shear Walls

The systematic analysis and design of existing wood and light frame shear walls, presented in the *Guidelines*, is a significant change from present design methodology. Shear walls with the same wall coverings, but of different lengths, are no longer considered to have equal capacity per unit length. Aspect ratio is taken into account, as is tie-down connection efficiency. Stiffnesses and deflections can be calculated. Walls of different construction can be compared on the basis of stiffness for distribution of loads. Wall deflections can be compared to diaphragm deflections for determination of diaphragm flexibility. Moreover, a larger wall assembly can be tested or modeled and used in place of the typical isolated, rectangular shear wall for design. A better, more accurate understanding and analysis of shear walls in buildings will result. Shear walls should be designed on the basis of performance; the *Guidelines* will provide the engineer with a more realistic understanding of shear wall performance.

Existing wood frame shear wall types addressed in this section include wood or metal stud walls with various kinds of sheathing. The sheathing generally defines the shear wall. The common existing sheathings are horizontal or diagonal lumber, horizontal or vertical wood siding, structural panels including plywood, stucco, gypsum plaster on various kinds of lath, various gypsum and wood panels, and combinations of various sheathings. Also included in this section are stud walls with various kinds of braces, and braced frames.

Standard test procedures need to be developed to replicate existing conditions as much as possible. These tests should provide the data needed to determine the strength capacities, stiffnesses, and governing of critical components of wood frame assemblies with various aspect ratios. See SEAOSC (1995) for a draft of a proposed testing standard.

C8.4.1 Types of Light Frame Shear Walls

No commentary is provided for this section.

C8.4.2 Light Gage Metal Frame Shear Walls

No commentary is provided for this section.

C8.4.3 Knee-Braced and Miscellaneous Timber Frames

C8.4.3.1 Knee-Braced Frames

No commentary is provided for this section.

C8.4.3.2 Rod-Braced Frames

These frames act as vertical trusses to resist lateral loads. Typically, the rods act only in tension. Once the capacity of the connection is determined, the elongation of the rods, as well as the movement in the connection of the rod to the wood frame, need to be investigated along with the other joints to establish the strength and stiffness of the frame.

C8.4.4 Single Layer Horizontal Lumber Sheathing or Siding Shear Walls

C8.4.4.1 Stiffness for Analysis

Very little is known about the stiffness of single layer horizontal lumber sheathing or siding shear walls. No cyclic test data for this assembly were found. Some indications of stiffness were derived from one dynamic diaphragm test that was studied. The shear wall stiffness presented in the *Guidelines* is surmised from the limited information available and is probably conservative. Single layer horizontal lumber sheathing or siding is very flexible and will experience degradation of stiffness and shear strength capacity when stressed beyond its yield capacity. The aspect ratio (height-to-length) of the shear wall may be the greatest determining factor of the wall's flexibility. Cut-in braces and diagonal blocking will provide some additional stiffness at lower force levels, but will probably not affect performance at yield or ultimate strength. More research is needed to more accurately determine the behavior of these shear walls. Where the height-to-width ratio exceeds 1.0, the wall should be disregarded as part of the lateral-force-resisting system.

C8.4.4.2 Strength Acceptance Criteria

For vertical diaphragms, the moment capacity—formed by the nail couple where each board crosses a stud—is obtained by multiplying the lateral strength for the size of nail used by the distance between nails in the same board. The resisting moment furnished by the nail couple is the moment per board per stud spacing. Multiplying the moment due to the nail couple by the number of boards in the height of the diaphragm gives the total moment capacity per stud spacing. Dividing the moment capacity of the nail couples by the wall height gives the lateral load capacity in pounds per stud spacing. This can be converted to pounds per linear foot by dividing by the stud spacing in feet. The allowable shear load per foot can then be multiplied by a factor of 2.8 to obtain the yield capacity of the shear wall. Details such as nailing and width of the individual sheathing boards will determine the capacity of the element. Connections to elements above and below will also determine the performance and force-displacement characteristics. The size of studs, plates, and boundary members will affect performance. Additional information on nail couple analysis can be found in the *Western Woods Use Book* published by the Western Wood Products Association (WWPA, 1983).

This analysis has not been compared to cyclic test results and may not be applicable. The indications from the one dynamic diaphragm test performed were used to provide the estimated yield strength presented in the *Guidelines*. Additional research is needed for greater accuracy.

C8.4.4.3 Deformation Acceptance Criteria

Accurate shear values and the associated deformations for single layer horizontal lumber sheathing or siding have not been developed. However, single layer horizontal lumber sheathing or siding will most likely be too flexible to limit displacements and associated damage. It is not recommended that these shear walls be used to resist lateral loads at higher Performance Levels such as Immediate Occupancy. Where lateral loads on these walls are low, attaining a Life Safety Performance Level is possible. This should be reviewed on a case-by-case basis, because the magnitude of deformation acceptable at Life Safety and Immediate Occupancy Performance Levels is dependent on acceptable deformations of other structural and nonstructural elements.

C8.4.4.4 Connections

No commentary is provided for this section.

C8.4.5 Diagonal Lumber Sheathing Shear Walls

C8.4.5.1 Stiffness for Analysis

The stiffness of diagonal lumber sheathed shear walls has not been determined. As of this writing, no cyclic test data have been found. However, there is some cyclic test data available for horizontal diagonally sheathed diaphragms. These few tests indicate a significant increase in stiffness over single layer horizontal sheathed shear walls. Also, displacements should be significantly less for diagonally sheathed shear walls. Deflections are still large when compared to plywood shear walls. The stiffness values presented in the *Guidelines* are estimated. More research is needed to determine the behavior of these shear walls.

C8.4.5.2 Strength Acceptance Criteria

Cyclic tests of diagonally sheathed shear walls are not available. The yield capacity presented in the *Guidelines* is estimated, based on general information and the limited relationships that can be inferred from the few cyclic diaphragm tests that have been conducted involving diagonal sheathing.

In general, diagonally sheathed shear walls have greater yield capacity than single layer horizontal sheathed shear walls because of the triangulated structural system. The lateral forces are resisted by tension and compression in the sheathing boards, and, because the sheathing boards are laid on a 45-degree angle, forces at the end members are also on a 45-degree angle to the end members. Nailing at the ends of the sheathing boards must be sufficient to transfer the desired force from the sheathing to the end members. The outward and inward thrust from the sheathing boards in compression or in tension introduces bending stresses in the perimeter members. Where shear stresses are high, special consideration must be given to the design of perimeter members for bending forces. The attachments of the perimeter members at the corners of the shear wall are also important. Sufficient attachment must be provided to prevent the perimeter members from separating at the corners due to the bending forces. Details such as the nailing and width of the individual sheathing boards will determine the capacity of the component or element. The sizes of studs, plates, and boundary members will also affect performance.

C8.4.5.3 Deformation Acceptance Criteria

Allowable shear values and the associated deformations for diagonally sheathed shear walls have not been fully developed, due to the lack of cyclic test data. Diagonally sheathed shear walls are suitable where lower Performance Levels are desired. Where a higher Performance Level such as Immediate Occupancy is desired, diagonally sheathed shear walls may or may not provide suitable shear strength, and stiffness depending on load levels. Great care is recommended if these shear walls are used to resist lateral loads at higher Performance Levels such as Immediate Occupancy. The magnitude of deformation acceptable at the Life Safety and Immediate Occupancy Performance Levels is dependent on acceptable deformations of other structural and nonstructural elements.

C8.4.5.4 Connections

No commentary is provided for this section.

C8.4.6 Vertical Wood Siding Shear Walls

C8.4.6.1 Stiffness for Analysis

The stiffness of vertical wood siding shear walls has not been determined. As of this writing, no cyclic test data have been found. Vertical wood siding is very flexible and will experience degradation of stiffness and shear capacity when stressed beyond its yield capacity. The stiffness value presented in the *Guidelines* is a best estimate. More research is needed to determine the behavior of these shear walls.

C8.4.6.2 Strength Acceptance Criteria

Cyclic tests of vertical wood siding shear walls are not available. The yield capacity presented in the *Guidelines* is estimated.

Vertical wood siding develops lateral capacity by nail couples in much the same manner as single layer horizontal wood siding. Since vertical boards are nailed to blocking between the studs, the spacing of the blocking will determine the capacity. Otherwise, the discussion of strength acceptance for horizontal wood sheathing and siding applies equally to vertical siding.

C8.4.6.3 Deformation Acceptance Criteria

Allowable shear values and associated deformations for vertical wood siding have not been fully developed due to the lack of cyclic test data. As of this writing, it is not recommended that these walls be used to resist lateral loads at higher Performance Levels such as Immediate Occupancy, even at low load levels.

C8.4.6.4 Connections

No commentary is provided for this section.

C8.4.7 Wood Siding over Horizontal Sheathing Shear Walls

C8.4.7.1 Stiffness for Analysis

Very little is known about the stiffness of wood siding over horizontal sheathing; no cyclic test data were found. Some indications of stiffness can be derived from one dynamic horizontal diaphragm test (ABK, 1981). The shear wall stiffness presented in the *Guidelines* is estimated.

This is a very common type of construction for older existing buildings. Compared to single layer horizontal sheathed shear walls, some additional stiffness—due to the wood siding—is expected for these shear walls. Greater stiffness occurs where the siding layers are at right angles to each other. More research is needed to determine the behavior of these shear walls.

C8.4.7.2 Strength Acceptance Criteria

Cyclic tests of these shear walls are not available. The yield capacity presented in the *Guidelines* is estimated, based on the general information noted and the limited relationships that can be inferred from the few available cyclic diaphragm tests involving two layers of transverse sheathing.

Typically, the horizontal sheathing will take most of the load, as it is the stiffer element. Some additional strength from lamination of siding and sheathing may occur, especially with vertical siding over horizontal sheathing. Details such as nailing and width of the individual sheathing boards will determine the capacity of the component or element. Connections to components or elements above and below will also determine the performance and force-displacement characteristics. The size of studs, plates, and boundary members will affect performance. More research is needed to determine the behavior of these shear walls.

C8.4.7.3 Deformation Acceptance Criteria

Allowable shear values and associated deformations for wood siding over horizontal sheathing have not been fully developed, due to the lack of cyclic test data. Great care is recommended if these shear walls are used to resist lateral loads at higher Performance Levels such as Life Safety or Immediate Occupancy. Wood siding over horizontal sheathing will probably be too flexible to limit displacements and associated damage to an acceptable level, except in areas of low seismicity. The magnitude of deformation acceptable at Life Safety and Immediate Occupancy Performance Levels is dependent on acceptable deformations for other structural and nonstructural elements.

C8.4.7.4 Connections

No commentary is provided for this section.

C8.4.8 Wood Siding over Diagonal Sheathing Shear Walls

C8.4.8.1 Stiffness for Analysis

Very little is known about the stiffness of wood siding over diagonal sheathing. As of this writing, no cyclic test data were found. Some indications of stiffness could be derived from one dynamic horizontal diaphragm test (ABK, 1981). The shear wall stiffness presented in the *Guidelines* is an estimate.

The cyclic test data available for horizontal diaphragms indicate that a significant increase in stiffness could be expected over single layer diagonally sheathed shear walls. The outside layer of wood siding has a stiffening effect on the diagonal sheathing and counteracts the bending effects in the edge members. As previously stated, these bending effects are present in single layer diagonally sheathed shear walls and can cause decreased stiffness in the shear wall. More research is needed to determine the behavior of these shear walls.

C8.4.8.2 Strength Acceptance Criteria

Cyclic tests of wood siding over diagonally sheathed shear walls are not available. The yield capacity presented in the *Guidelines* is estimated, based on the general information noted below and the limited relationships that can be inferred from the one dynamic horizontal diaphragm test involving straight sheathing over diagonal sheathing.

Typically, the diagonal sheathing would take the load as the stiffer element until failure. Some additional strength from lamination of siding and sheathing certainly will occur. Tests from horizontal diaphragms with straight sheathing over diagonal sheathing suggest that this type of shear wall may be suitable for moderate to fairly high shear loads. For shear walls with wood siding over diagonal sheathing, the forces in the diagonal sheathing will produce bending in the perimeter members that is counteracted by the wood siding. This counteracting of force within the shear wall assembly may relieve the perimeter members of bending stresses. Because of this reduction of bending in the perimeter members, both the yield capacity and stiffness of the shear wall are increased over those of a diagonally sheathed shear wall. Detailing, such as nailing and width of the individual sheathing boards, will also determine the capacity of the component or element. The size of studs, plates, and boundary members will also affect performance.

C8.4.8.3 Deformation Acceptance Criteria

Allowable shear values and associated deformations for wood siding over diagonally sheathed shear walls have not been fully developed, due to the lack of cyclic test data. Wood siding over diagonally sheathed shear walls may be used for higher Performance Levels such as Life Safety and Immediate Occupancy, due to the increase in yield capacity and stiffness. Full-scale mock-up cyclic load tests are recommended if these shear walls are used to resist lateral loads at these higher Performance Levels.

C8.4.8.4 Connections

No commentary is provided for this section.

C8.4.9 Structural Panel or Plywood Panel Sheathing Shear Walls

C8.4.9.1 Stiffness for Analysis

Deflections for structural panel or plywood panel sheathed shear walls can be calculated according to the methods shown in Section 8.4.9 of the *Guidelines.* These methods are based on the *Uniform Building Code* (UBC) (ICBO, 1994a), and various APA publications. A significant amount of monotonic shear wall testing has been performed by the APA. In addition, some cyclic loading test data are available for plywood panel sheathing and structural panel shear walls. However, because there is no standard testing procedure or data recording protocol for cyclic loading tests, much of the information supplied in the tests is incomplete. The stiffness of wood structural shear walls is affected by the thickness, the height-to-length ratio, the nailing pattern, the blocking, and the tie-downs of panels, as well as other factors. The stiffness cannot be determined with great accuracy. More cyclic testing is needed to determine the behavior of these shear walls.
Equation 8-2 is taken from Section 23.223 of the UBC (ICBO, 1994a) with (h/b) modifier added to the deflection component d_a. The accuracy of this equation needs confirmation by additional research. Of particular concern is deflection due to anchorage details; the effect on wall performance can be significant and may overshadow all other factors. At present there is very limited information on d_a values.

C8.4.9.2 Strength Acceptance Criteria

Tables with allowable shear values for various types of wood structural panel shear walls have been published by a number of building code agencies, and industry organizations such as the APA. These tables contain allowable shear values that are derived from monotonic tests.

A standard cyclic test would be valuable to determine allowable cyclic shear values for these shear walls. Presently, the ultimate cyclic capacity can be estimated as 80% of the static ASTM-E72 ultimate as determined by APA tests. This estimate is only applicable to walls with aspect ratios of 1.0 or less. There are some tests from Japan (Yasumura, 1992) that support this estimate. Detailing, such as nailing and thickness of panels, will determine the capacity of the component or element. Connections to components or elements above and below the wall will also determine performance and force-displacement characteristics. The size of studs, plates, and boundary members will also affect performance. Components and elements with openings will be more flexible. Equation 8-4 is taken from Yasumura (1992).

Wood structural panel shear walls have a broad range of shear capacities and stiffnesses; therefore, these shear walls are suitable for a wide range of Performance Levels. Shear wall capacity and stiffness must be compatible with the desired Performance Level and the level of acceptable damage. At higher Performance Levels such as Immediate Occupancy, wood structural panel shear walls are capable of higher yield capacities with decreased displacements, due to higher stiffness as compared to other types of shear walls. Figure 8-1 was constructed using: (1) adjusted available values, (2) equations for deflection (from Section 23.223 of the 1994 UBC), (3) Yasumura (1992), and (4) a comparison of the backbone curves from test results with constructed backbone curves. Future research should provide a more accurate method for constructing a backbone curve. Future research should also provide more information on larger wall assemblies, with various size openings.

C8.4.9.3 Deformation Acceptance Criteria

No commentary is provided for this section.

C8.4.9.4 Connections

No commentary is provided for this section.

C8.4.10 Stucco on Studs, Sheathing, or Fiberboard Shear Walls

C8.4.10.1 Stiffness for Analysis

The stiffness of stucco shear walls has not been determined. As of this writing, no cyclic test data were found, and therefore no shear wall stiffness has been determined. The stiffness given in the *Guidelines* is estimated based on the following information. Stucco on studs is brittle and will experience degradation of stiffness and shear capacity when stressed beyond its yield capacity. The aspect ratio of the shear wall may control the wall's flexibility. More research is needed to determine the behavior of these shear walls.

C8.4.10.2 Strength Acceptance Criteria

The performance of stucco shear walls may have two stages. In the first stage, before yielding, the stucco shear wall will be stiff, similar to a concrete wall. For the second stage, the stucco shear wall will be flexible from yielding and wire deformation. The capacity given in the *Guidelines* is estimated for the first stage of performance. Detailing, such as nailing or stapling of the stucco nettings, will effect the capacity of the component or element. The size of studs, plates, and boundary members will also affect performance. Components or elements with openings will be more flexible. Connections to elements above and below will also determine performance and force-displacement characteristics.

C8.4.10.3 Deformation Acceptance Criteria

A stucco shear wall is expected to have a higher yield capacity than a gypsum plaster wall and, due to the brittle nature of stucco, a smaller elastic range than a plywood wall. Allowable shear values and associated deformations for stucco have not been developed, due to the lack of cyclic test data. Stucco shear walls should be considered to be brittle.

C8.4.10.4 Connections

No commentary is provided for this section.

C8.4.11 Gypsum Plaster on Wood Lath Shear Walls

C8.4.11.1 Stiffness for Analysis

Very little is known about the stiffness of gypsum plaster on wood lath. As of this writing, no cyclic test data were found, and therefore no shear wall stiffness could be determined. The stiffness given in the *Guidelines* is an estimate. Gypsum plaster on wood lath is relatively stiff until the plaster cracks; after that the wall becomes more flexible. Cut-in braces and diagonal blocking will provide some additional stiffness at lower force levels, but will not affect performance at yield or ultimate strength. More research is needed to determine the behavior of these shear walls.

C8.4.11.2 Strength Acceptance Criteria

Cyclic tests of gypsum plaster on wood lath are not available. The yield capacity presented in the *Guidelines* is an estimate. The strength of the plaster probably governs the capacity. Detailing, such as nailing, may have some influence in determining the capacity of the component or element. After the plaster cracks, strength is reduced and flexibility will increase.

C8.4.11.3 Deformation Acceptance Criteria

Due to the lack of cyclic test data, allowable shear values and associated deformations for gypsum plaster on wood lath have not been fully developed. These shear walls are not recommended to resist lateral loads at higher Performance Levels such as Immediate Occupancy. Gypsum plaster on wood lath will most likely be too flexible after the plaster cracks to limit displacements and associated damage to an acceptable level.

C8.4.11.4 Connections

No commentary is provided for this section.

C8.4.12 Gypsum Plaster on Gypsum Lath Shear Walls

C8.4.12.1 Stiffness for Analysis

The stiffness of gypsum plaster on gypsum lath has not been fully determined. As of this writing, no shear wall stiffness could be determined, because no cyclic test data were found. The stiffness given in the *Guidelines* is an estimate. Gypsum plaster on gypsum lath should be relatively stiff until the plaster cracks; after that the wall becomes more flexible. Cut-in braces and diagonal blocking will provide some additional stiffness at lower force levels, but will not affect performance at yield or ultimate strength. More research is needed to determine the behavior of these shear walls.

C8.4.12.2 Strength Acceptance Criteria

Cyclic tests of gypsum plaster on gypsum lath are not available. The yield capacity presented in the *Guidelines* is an estimate. The strength of the combined plaster and lath will probably govern the capacity. Detailing such as nailing should have some influence in determining the capacity of the component or element. After the plaster and lath crack, strength is reduced and flexibility will increase.

C8.4.12.3 Deformation Acceptance Criteria

Allowable shear values and associated deformations for gypsum plaster on gypsum lath have not been fully developed, due to the lack of cyclic test data. These walls are not recommended for resisting lateral loads at higher Performance Levels such as Immediate Occupancy.

C8.4.12.4 Connections

No commentary is provided for this section.

C8.4.13 Gypsum Wallboard Shear Walls

C8.4.13.1 Stiffness for Analysis

Cyclic testing for gypsum wallboard is available from various sources. However, the testing methods differed and results were reported differently. One of the sources is Report No. UCB/EERC-85/06 (Oliva, 1986). The walls in this test were one-sided, without either tie-downs at the end of the walls or dead load applied to the top of the wall to simulate usual conditions. As in the test, most gypsum wallboard shear walls do not have tie-downs at the ends of the walls. If an actual wall frames into a corner at each end of the wall and the aspect ratio is low, a higher ultimate capacity should be expected. Both additional research on the available data and new testing are needed. The effect of the aspect ratio has not been addressed, but may determine the wall's flexibility and mode of failure. The report cited above showed that glued gypsum wallboard panels were much stiffer and stronger, but less ductile. Gypsum wallboard will experience degradation of stiffness and shear capacity when stressed beyond its yield capacity. Cut-in braces and diagonal blocking will provide some additional stiffness at lower force levels, but will probably not affect performance at yield or ultimate strength. As with other wall assemblies, more research is needed to determine the behavior of these shear walls. In the interim, an estimated stiffness is included in the *Guidelines*.

C8.4.13.2 Strength Acceptance Criteria

The strength of the gypsum wallboard, and detailing such as nailing, should have some influence in determining the capacity of the element. The capacity given in the *Guidelines* is an estimate; a more accurate capacity should be available once a standard test method is developed. After the wallboard cracks, or the nails enlarge the holes in the boards, strength is reduced and flexibility increases.

C8.4.13.3 Deformation Acceptance Criteria

The tests available indicate very little deflection can be tolerated without enlargement of nail holes. These shear walls are not recommended for resisting lateral loads at higher Performance Levels such as Immediate Occupancy.

C8.4.13.4 Connections

No commentary is provided for this section.

C8.4.14 Gypsum Sheathing Shear Walls

C8.4.14.1 Stiffness for Analysis

See Section C8.4.13.1.

C8.4.14.2 Strength Acceptance Criteria

See Section C8.4.13.2.

C8.4.14.3 Deformation Acceptance Criteria

See Section C8.4.13.3.

C8.4.14.4 Connections

No commentary is provided for this section.

C8.4.15 Plaster on Metal Lath Shear Walls

C8.4.15.1 Stiffness for Analysis

The stiffness of plaster on metal lath has not been fully determined. At this time, no cyclic test data were found, and therefore no shear wall stiffness could be determined. The stiffness given in the *Guidelines* is an estimate. Plaster on metal lath is relatively brittle and will experience degradation of stiffness and shear capacity when stressed beyond its yield capacity.

Braces and diagonal straps will provide some additional stiffness at lower force levels, but will probably not affect performance at yield or ultimate strength. More research is needed to determine the behavior of these shear walls.

C8.4.15.2 Strength Acceptance Criteria

As with stucco on studs, the performance of plaster on metal lath may have two stages. In the first stage, before yielding, the plaster on metal lath shear wall will be stiff, similar to a concrete wall. For the second stage, the plaster on metal lath shear wall will be flexible from yielding and wire deformation. The capacity given in the *Guidelines* is a best estimate for the first stage of performance. Detailing, such as nailing of the metal lath, will affect the capacity of the component or element. The size of studs, plates, and boundary members will affect performance. Components or elements with openings will be more flexible. Connections to elements above and below will also determine performance and force-displacement characteristics.

C8.4.15.3 Deformation Acceptance Criteria

A plaster on metal lath shear wall is expected to have a higher yield capacity than plaster by itself and, due to the brittle nature of plaster, a smaller elastic range than plywood panel sheathed shear walls. Allowable shear values and associated deformations for plaster on metal lath have not been developed, due to the lack of cyclic test data. Plaster on metal lath shear walls will be too brittle to provide for higher Performance Levels except in areas of low seismicity.

C8.4.15.4 Connections

No commentary is provided for this section.

C8.4.16 Horizontal Lumber Sheathing with Cut-In Braces or Diagonal Blocking Shear Walls

C8.4.16.1 Stiffness for Analysis

See Section C8.4.4.1.

C8.4.16.2 Strength Acceptance Criteria

See Section C8.4.4.2.

C8.4.16.3 Deformation Acceptance Criteria

See Section C8.4.4.3.

C8.4.16.4 Connections

No commentary is provided for this section.

C8.4.17 Fiberboard or Particleboard Sheathing Shear Walls

C8.4.17.1 Stiffness for Analysis

See Section C8.4.9.1.

C8.4.17.2 Strength Acceptance Criteria

See Section C8.4.9.2.

C8.4.17.3 Deformation Acceptance Criteria

See Section C8.4.9.3.

C8.4.17.4 Connections

No commentary is provided for this section.

C8.4.18 Light Gage Metal Frame Shear Walls

C8.4.18.1 Plaster on Metal Lath

See Section C8.4.15.1.

C8.4.18.2 Gypsum Wallboard

See Section C8.4.13.

C8.4.18.3 Plywood or Structural Panels

No commentary is provided for this section.

C8.5 Wood Diaphragms

There are a number of resource documents pertaining to wood diaphragms. Various APA publications and research reports contain more detailed information on analysis methods and testing data for wood diaphragms. *Guidelines for the Design of Horizontal Wood Diaphragms* (ATC, 1981) also contains valuable information on the design and detailing of wood diaphragms. The National Science Foundation (NSF) has sponsored static and dynamic tests of wood diaphragms, performed by the joint venture ABK. This document is entitled *Methodology for Mitigation of Seismic Hazards in Existing Unreinforced Masonry Buildings: Diaphragm Testing* (ABK, 1981).

C8.5.1 Types of Wood Diaphragms

No commentary is provided for this section.

C8.5.2 Single Straight Sheathed Diaphragms

C8.5.2.1 Stiffness for Analysis

Deflection of straight sheathed diaphragms cannot be calculated by rational methods of analysis. The diaphragm shear stiffness has been determined from testing results of typical straight sheathed diaphragms, which are very flexible and experience degradation of stiffness and shear capacity when stressed beyond their yield capacity and at high deflections. More research is needed to determine diaphragm behavior where forces act parallel to the sheathing. Shear capacity parallel to the sheathing boards is dependent on shear transfer between sheathing boards by nails into the framing members.

C8.5.2.2 Strength Acceptance Criteria

For horizontal diaphragms, the moment capacity, formed by the nail couple where each board crosses a joist, is obtained by multiplying the lateral strength for the size of nail used, by the distance between nails in the same board. Dividing this moment by the joist spacing gives the end reaction or shear load per board width. This in turn is multiplied by the ratio of the net width of the board to one foot, which results in the allowable end reaction or shear load in pounds per linear foot for the diaphragm. The allowable shear load per foot can be multiplied by a factor of 2.8 to obtain the yield capacity of the diaphragm. See ATC (1981) for a discussion on calculating the allowable shear capacity of straight sheathed diaphragms.

C8.5.2.3 Deformation Acceptance Criteria

Allowable shear values and associated deformations for straight sheathed diaphragms have been developed for seismic rehabilitation to the Collapse Prevention Performance Level. Great care should be exercised if these diaphragms are used to resist lateral loads at higher Performance Levels such as Immediate Occupancy. Straight sheathed diaphragms will most likely be too flexible to limit displacements and associated damage to an acceptable level, except in areas of low seismicity. The magnitude of deformation acceptable at Life Safety and Immediate Occupancy Performance Levels is dependent on acceptable deformations for other structural and nonstructural components and elements.

C8.5.2.4 Connections

No commentary is provided for this section.

C8.5.3 Double Straight Sheathed Wood Diaphragms

C8.5.3.1 Stiffness for Analysis

Information on force versus displacement curves for double straight sheathed diaphragms has not been located. Further research on the response of double straight sheathed diaphragms would be valuable.

C8.5.3.2 Strength Acceptance Criteria

Shear capacity is dependent on the nailing of the diaphragm. This type of diaphragm is suitable for moderate to high shear loads. Placement of the second layer of straight sheathing will provide a significant increase in both the yield capacity and stiffness of the diaphragm over that of a single sheathed diaphragm. Further research needs to be done on this type of diaphragm to obtain more information on the yield shear capacity.

C8.5.3.3 Deformation Acceptance Criteria

Because of the increased yield capacity and stiffness over many other types of wood diaphragms, double sheathed diaphragms may be compatible with higher Performance Levels such as Life Safety and Immediate Occupancy, where shear demands are low. Diaphragm displacements will need to be compatible with other building materials and the desired Performance Level.

C8.5.3.4 Connections

No commentary is provided for this section.

C8.5.4 Single Diagonally Sheathed Wood Diaphragms

C8.5.4.1 Stiffness for Analysis

Force-versus-displacement curves for these diaphragms have been developed as part of various testing programs. These testing programs indicated a significant increase in stiffness over straight sheathed diaphragms. While displacements will be significantly less than for straight sheathed diaphragms, displacements will still be large. Diaphragm deflections cannot be calculated by rational analysis, and will need to be predicted using the procedures of Section 8.5.4 of the *Guidelines*.

C8.5.4.2 Strength Acceptance Criteria

Diagonally sheathed diaphragms have greater yield shear capacity than straight sheathed diaphragms because of the triangulated structural system. The lateral forces are resisted by tension and compression in the sheathing boards, and, because the sheathing boards are laid on a 45-degree angle, forces at the end members are also on a 45-degree angle to the end members. Nailing at the ends of the sheathing boards must be sufficient to transfer the desired force from the sheathing to the end members. The shear capacity of the diaphragm is the component of the force that is parallel to the end members, which is transferred by the end nailing at each board. The outward and inward thrust from sheathing boards in compression or in tension introduces bending stresses in the perimeter members, in addition to the axial stresses accruing from their position as flange or chord members in the diaphragm. Special consideration must be taken to design the perimeter members for bending forces. The attachment of the perimeter members at the corners of the diaphragm is also important. Sufficient attachment must be provided to prevent the perimeter members from separating at the corners due to the bending forces.

C8.5.4.3 Deformation Acceptance Criteria

Because displacements will be significant for diagonally sheathed diaphragms, they are best suited where lower Performance Levels such as Collapse Prevention are desired. Where higher Performance Levels such as Immediate Occupancy are desired, diagonally sheathed diaphragms may not provide suitable shear strength and stiffness.

C8.5.4.4 Connections

No commentary is provided for this section.

C8.5.5 Diagonal Sheathing with Straight Sheathing or Flooring Above Wood Diaphragms

C8.5.5.1 Stiffness for Analysis

Diaphragm testing programs by ABK (1981) and others indicate a significant increase in stiffness for these diaphragms over single sheathed diaphragms. The upper layer of straight sheathing or flooring has a significant stiffening effect in the diaphragm and counteracts the bending effects in the diaphragm edge members that are present in single diagonally sheathed diaphragms.

C8.5.5.2 Strength Acceptance Criteria

Shear capacity is dependent on the nailing of the diaphragm. This type of diaphragm is suitable for moderate to high shear loads. For diaphragms with diagonal sheathing and straight sheathing or flooring above, the forces in the diagonal sheathing that produce bending in the perimeter members are resisted by the straight sheathing. This cornerstone relieves the perimeter members of bending stresses, leaving only the axial stresses from chord action. Because of this reduction of stress in the perimeter members, both the yield capacity and stiffness of the diaphragm are greatly increased over those of a single sheathed diaphragm.

C8.5.5.3 Deformation Acceptance Criteria

Because of the increased yield capacity and stiffness over many other types of wood diaphragms, diagonally sheathed diaphragms with straight sheathing or flooring above may be more compatible with higher Performance Levels such as Life Safety and Immediate Occupancy. Diaphragm displacements will need to be compatible with other building materials and the desired Performance Level.

C8.5.5.4 Connections

No commentary is provided for this section.

C8.5.6 Double Diagonally Sheathed Wood Diaphragms

C8.5.6.1 Stiffness for Analysis

Testing and related force-versus-displacement information for double diagonally sheathed diaphragms is limited, but the diaphragm will respond similarly to diagonally sheathed diaphragms with straight sheathing or flooring above. Double sheathed diaphragms will be significantly stiffer than single sheathed diaphragms. Further research on the response of double diagonally sheathed diaphragms would be valuable.

C8.5.6.2 Strength Acceptance Criteria

Shear capacity is dependent on the nailing of the diaphragm. When double diagonal sheathing is used, the outward forces on the perimeter members from that portion of the sheathing in compression, are counteracted by the inward forces from that portion of the sheathing in tension. This counteracting of forces within the sheathing assembly relieves the perimeter members of bending stresses, leaving only the axial stresses from their chord action. Because of this reduction of bending in the perimeter members, both the yield capacity and stiffness of the diaphragm are increased over those of a single sheathed diaphragm.

C8.5.6.3 Deformation Acceptance Criteria

Because of the increased yield capacity and stiffness over many other types of wood diaphragms, double diagonally sheathed diaphragms are more compatible with higher Performance Levels such as Life Safety and Immediate Occupancy. Diaphragm displacements will need to be compatible with other building materials and the desired Performance Level.

C8.5.6.4 Connections

No commentary is provided for this section.

C8.5.7 Wood Structural Panel Sheathed Diaphragms

C8.5.7.1 Stiffness for Analysis

Deflections for wood structural panel diaphragms can be calculated according to the accepted methods shown in Section 8.5.7 of the *Guidelines*, which are based on ATC (1981), UBC (ICBO, 1994a), and various APA publications. A significant amount of monotonic diaphragm testing has been performed by APA and other agencies. Some dynamic testing was performed during the ABK (1981) testing program. Testing programs have indicated that wood structural panel diaphragms that are blocked and chorded are stiffer and have a higher shear capacity than unblocked or unchorded wood structural panel diaphragms and many other types of wood diaphragms. Even with this increase in stiffness, wood structural panel diaphragms are still considered to be flexible diaphragms in most cases. In cases with low diaphragm length-to-width ratios and fairly flexible vertical lateral-force-resisting elements, wood structural panel diaphragms may need to be considered as rigid or semi-rigid diaphragms.

C8.5.7.2 Strength Acceptance Criteria

Tables with allowable shear values for various types of wood structural panel diaphragms have been published by a number of building code agencies and industry organizations such as APA. These diaphragms have a fairly broad range of allowable shear capacities. Yield capacities for wood structural panel diaphragms can be estimated by multiplying the allowable shear values by a factor of 2.1 for chorded diaphragms and 1.75 for unchorded diaphragms. The factor 2.1 is used in lieu of a 2.8 factor because a load duration factor of 1.33 is included in the National Design Specification (AF&PA, 1991) value.

C8.5.7.3 Deformation Acceptance Criteria

Wood structural panel diaphragms have a broad range of shear capacity and stiffness, so the diaphragms may be suitable for a broad range of Performance Levels. Diaphragm shear capacity and stiffness must be compatible with the desired Performance Level and the level of allowable damage. At higher Performance Levels such as Life Safety and Immediate Occupancy, wood structural panel diaphragms are capable of higher yield capacities with decreased displacements, due to higher stiffness.

C8.5.7.4 Connections

No commentary is provided for this section.

C8.5.8 Wood Structural Panel Overlays On Straight or Diagonally Sheathed Diaphragms

C8.5.8.1 Stiffness for Analysis

Testing of these diaphragms has been performed by APA as well as ABK (1981). The wood structural panel overlay creates a very significant increase in diaphragm strength and stiffness when placed over a straight sheathed diaphragm. When a new wood structural panel overlay is placed over a diagonally sheathed diaphragm, the increase in strength and stiffness will not be proportional to that achieved for a straight sheathed diaphragm, but will still be significant. This is due to the initial stiffness of the diagonally sheathed diaphragm being higher than that of the straight sheathed diaphragm.

C8.5.8.2 Strength Acceptance Criteria

The allowable shear capacity for wood structural panel overlays has been limited by the *Uniform Code for Building Conservation* (UCBC) (ICBO, 1994b) to 225 pounds per foot for unblocked diaphragms, regardless of the nailing used to attach the plywood to the supporting framing members. For blocked wood structural panel diaphragms, the UCBC limits the allowable shear capacity of the overlay to 75% of the value specified for the horizontal diaphragm shear table of the UBC (ICBO, 1994a). The reason for the lower values is that the nail sizes commonly used for nailing of wood structural panels have required embedment lengths that exceed the board thickness of the existing sheathing. Splitting of the existing sheathing boards is also common, especially at the closely spaced edge nailing at the perimeter of the wood structural panels.

The values given for wood structural panels applied over existing sheathing boards are for that assembly only. If the existing boards are removed and the wood structural panels are applied directly to the existing framing members, shear values discussed in Section 8.5.7 should be used. The increase in allowable shear capacity will provide a moderate to high increase in diaphragm yield capacity over that provided by the existing sheathing. Diaphragm yield capacity and displacement requirements at various Performance Levels will need to be coordinated with the force-versus-displacement curves to ensure compatibility with the type of construction of the existing components and elements in the building.

C8.5.8.3 Deformation Acceptance Criteria

Wood structural panel overlays on existing sheathed diaphragms have a broad range of shear capacities and stiffnesses, so the diaphragms may be suitable for a broad range of Performance Levels. Diaphragm shear capacity and stiffness must be compatible with the desired Performance Level and allowable damage. At higher Performance Levels such as Life Safety and Immediate Occupancy, wood structural panel overlays over existing sheathed diaphragms may be capable of higher yield capacities with decreased displacements, due to higher stiffnesses.

C8.5.8.4 Connections

No commentary is provided for this section.

C8.5.9 Wood Structural Panel Overlays on Existing Wood Structural Panel Diaphragms

C8.5.9.1 Stiffness for Analysis

Some monotonic testing of these diaphragms has been performed by APA. Test results indicate that shear capacity and stiffness of an existing wood structural panel can be increased significantly by adding a new wood structural panel overlay over an existing diaphragm.

C8.5.9.2 Strength Acceptance Criteria

See Section C8.5.8.2.

C8.5.9.3 Deformation Acceptance Criteria

See Section C8.5.8.3.

C8.5.9.4 Connections

No commentary is provided for this section.

C8.5.10 Braced Horizontal Diaphragms

C8.5.10.1 Stiffness for Analysis

The stiffness of braced horizontal diaphragms can vary with different systems, but is most often flexible, with a long period of vibration. Classical deflection analysis procedures can be used to determine the stiffness of the horizontal truss. Length-to-width ratios of the truss system can have a significant effect on the stiffness of the horizontal truss. Lower length-to-width ratios will result in increased stiffness of the horizontal truss; higher length-to-width ratios will result in decreased stiffness of the horizontal truss. Distortion in the rod brace connection shall be incorporated into the truss deflection analysis.

C8.5.10.2 Strength Acceptance Criteria

The size and mechanical properties of the tension rods, compression struts, and connection detailing are all important to the yield capacity of the braced horizontal diaphragm. Standard truss analysis techniques can be used to determine the yield capacity of the braced horizontal diaphragm. Special attention is required at connections between different members of the truss system. Yield capacity of the connections will in many cases limit the yield capacity of the truss system. Connections that will develop the yield capacity of the truss members and reduce the potential for brittle failure are desired. If enhancement of existing braced horizontal diaphragms is required, classical truss analysis methods can be used to determine which members or connections require enhancement. Analysis of existing connections, and enhancement of connections with insufficient yield capacity, should be performed in a manner that will encourage yielding in the truss members rather than brittle failure in the truss connections.

C8.5.10.3 Deformation Acceptance Criteria

More flexible, lower-strength braced horizontal diaphragm systems may perform well for rehabilitation to the Collapse Prevention Performance Level. Upgrades to Life Safety or Immediate Occupancy Performance Levels will require proportional increases in yield capacity and stiffness to control lateral displacements. Displacements must be compatible with the type of construction supported by the horizontal truss system.

C8.5.10.4 Connections

No commentary is provided for this section.

C8.5.11 Effects of Chords and Openings in Wood Diaphragms

Static and dynamic diaphragm testing programs have indicated that wood diaphragms with chords are stiffer than comparable diaphragms without chords. Chords may not be required in the diaphragm for a lower Performance Level such as Collapse Prevention. Documents such as the UCBC and the ABK methodology do not require chords in the diaphragm in most cases. These documents are geared toward Collapse Prevention Performance Levels. Where higher Performance Levels such as Life Safety or Immediate Occupancy are desired, chords will usually be required to limit deflections, except in areas of low seismicity.

Care should be exercised in stiffening diaphragms by overlaying with new materials, adding new chords, or other methods. Increased stiffness in the diaphragm will result in a shorter period of vibration and an associated increase in lateral force on the diaphragm. Under some conditions this decreased period and increased force may not be desirable. If displacements are not critical to the performance of the diaphragm or supported wall elements, the diaphragm may actually perform better at the longer period with a lower dynamic force.

C8.6 Wood Foundations

C8.6.1 Wood Piling

The method of analyzing wood piles is based on past performance and is empirical in application. Environmental conditions, such as changes in the water table, can result in deterioration of piles with a resultant loss in capacity. The assumption of point of restraint or fixity of the pile for lateral load analysis is very subjective and will have a significant effect on the results of the analysis both for stress level and anticipated deflection. Battered piles can be analyzed for static resistance to base shear.

C8.6.2 Wood Footings

Wood is generally not used as a foundation material for permanent structures, although there are code-approved pressure-treated wood systems for the foundations of small residential structures, which have been used in recent years in some areas.

C8.6.3 Pole Structures

Pole type structures, as well as structures constructed above grade on post or pole supports, are used in some areas of the country to reduce flood or storm damage, or accommodate sloping or irregular terrain. If not properly designed and detailed, pole-supported structures can be at high risk under seismic loading.

The pole structure is generally analyzed as a braced frame; it resists lateral loads by both the cantilever action of the poles embedded into the ground and by braced or sheathed frames in the superstructure. Like the wood piles, the stiffness of the structure is dependent on the character of the ground, fixity, and distortion of the soil into which the pole is founded.

C8.7 Definitions

In addition to the *Guidelines* listings, additional terms and descriptions can be found in standard construction dictionaries or encyclopedias. See Section C8.9.

C8.8 Symbols

The symbols used are generally in the form used in the reference material.

C8.9 References

In addition to the following references, many Canadian standards could be used to good advantage in the evaluation of existing buildings and possible upgrading methodologies.

ABK, 1981, *Methodology for Mitigation of Seismic Hazards in Existing Unreinforced Masonry Buildings: Diaphragm Testing*, Agbabian-Barnes-Kariotis Joint Venture Topical Report No. ABK-03, El Segundo, California.

AF&PA, 1991, *National Design Specification for Wood Construction, Commentary*, American Forest & Paper Association, Washington, D.C.

AITC, 1994, *Timber Construction Manual*, American Institute of Timber Construction, John Wiley & Sons, New York, New York.

Anthony, R. W., and Bodig, J., 1990, "Applications of Stress Wave-Based Nondestructive Evaluation to Wood," *Proceedings, Nondestructive Evaluation of Civil Structures and Materials*, University of Colorado, Boulder, Colorado, pp. 257–275.

APA, 1983, *Plywood Diaphragms*, Research Report No. 138, American Plywood Association, Tacoma, Washington.

ASCE, 1975, *Wood Structures*: *A Design Guide and Commentary,* American Society of Civil Engineers, New York, New York.

ASCE, 1982, *Evaluation, Maintenance and Upgrading of Wood Structures, A Guide and Commentary,* American Society of Civil Engineers, New York, New York.

ASCE, 1994, *Load & Resistance Factor Design (LRFD) Standard for Engineered Wood Construction* (in progress), American Society of Civil Engineers, New York, New York.

ASTM, latest edition, Standards having the following numbers: D143, D196, D1761, D1860, D2555, D2915, E22, E72, F606, American Standards for Testing Materials, Philadelphia, Pennsylvania.

ATC, 1980, *The Home Builders Guide for Earthquake Design*, Report No. ATC-4, Applied Technology Council, Redwood City, California.

ATC, 1981, *Guidelines for the Design of Horizontal Wood Diaphragms*, Report No. ATC-7, Applied Technology Council, Redwood City, California.

BOCA, 1993, *The BOCA National Building Code*, Building Officials & Code Administrators International, Inc., Country Club Hills, Illinois.

Breyer, D. E., 1993, *Design of Wood Structures, Third Edition*, McGraw-Hill, Inc., New York, New York.

Brooks, H., 1976, *Illustrated Encyclopedic Dictionary of Building and Construction Terms*, Prentice-Hall, Inc., Inglewood Cliffs, New Jersey.

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.

Degenkolb, H. J., and Associates, 1968, *Design Notes and Criteria–Pole Type Building*, J. H. Baxter & Co., Long Beach, California.

Dolan, J. D., et al., 1994, *Sequential Phased Displacement Tests to Determine Short-Term Duration-of-Load Performance of Nail and Bolt Connections, Volume I: Summary Report*, Virginia Polytechnic Institute and State University, Blacksburg, Virginia.

FHA, 1975, *Pole House Construction*, *Second Edition*, Federal Housing Administration, Washington, D.C.

Hoyle, Jr., R. J., 1978, *Wood Technology in the Design of Structures*, Mountain Press Publishing, Missoula, Montana.

ICBO, *National Evaluation Reports*–Various Reports of Wood Materials, Manufactured Wood Products, Fasteners & Hardware, International Conference of Building Officials, Whittier, California.

ICBO, 1994a, *Uniform Building Code*, International Conference of Building Officials, Whittier, California.

ICBO, 1994b, *Uniform Code for Building Conservation*, International Conference of Building Officials, Whittier, California.

Jedrzejewski, M. J., "A Method for Determining the Allowable Strength of In-Place Wood Structural Members," *Building Performance: Function, Preservation, and Rehabilitation*, ASTM STP 901, edited by Davis, G., American Society for Testing and Materials, Philadelphia, Pennsylvania, pp. 136–151.

Loferski, J., Dolan, J., and Lang, E., 1996,"Non-Destructive Evaluation and Testing Methods in Wood Buildings," *Standards for Preservation & Rehabilitation*, West Conshohocken, Pennsylvania.

Oliva, M. G., 1985, *Rocking Behavior of Wood-Framed Gypsum Panels under Dynamic Load*, Report No. UCB/EERC-85/06, Earthquake Engineering Research Center, University of California at Berkeley, California.

Patterson, D., 1969, *Pole Building Design*, American Wood Preservers Institute, Chicago, Illinois.

RILEM, 1994, "Behavior of Timber Structures under Seismic Actions," *Materials & Structures*, London, United Kingdom, pp. 158–184.

Ross, R. J., and Pellerin, R. F., 1991, *Nondestructive Testing for Assessing Wood Members in Structures, A Review*, General Technical Report FLP-GRT-70, Forest Products Laboratory, Madison, Wisconsin.

SBCCI, 1991, *Standard Building Code*, Southern Building Code Congress International, Inc., Birmingham, Alabama.

Schierle, G. G., "Quality Control in Seismic Resistant Construction," *Report to the National Science Foundation*, Arlington, Virginia.

SEAOSC, 1995, *Standard Method of Cyclic (Reversed) Load Test for Shear Resistance of Framed Walls for Buildings* (draft), Structural Engineers Association of Southern California, Whittier, California.

Sherwood, G. E., 1986, "Technology of Preserving Wood Structures," *Building Performance: Function, Preservation, and Rehabilitation*, ASTM STP 901, edited by Davis, G., American Society for Testing and Materials, Philadelphia, Pennsylvania, pp. 121–135.

Simpson, 1996, *Wood Construction Connectors*, Catalog No. C-96, Simpson Company, San Leandro, California.

Somayaji, S., *Structural Wood Design*, West Publishing Company, St. Paul, Minnesota.

Stalnaker, J. J., and Harris, E. C., 1989, *Structural Design in Wood*, Van Nostrand Reinhold, New York, New York.

TECO, 1970, *Design Manual for TECO Timber Connector Construction*, Timber Engineering Company, Chevy Chase, Maryland.

TSM, 1992, "Seismic Design for Buildings," *Tri-Services Manual TM5-809-10 NAV FAC P 355*, AFM 88-3, Chapter 13, Washington, D.C.

USDA, 1987, *Wood as an Engineering Material*, Handbook 72, Forest Products Laboratory, U.S. Department of Agriculture, Madison, Wisconsin.

WWPA, 1983, *Western Woods Use Book*, Western Wood Products Association, Portland, Oregon.

Yasumura, M., 1992, *Mechanical Properties of Wood-Framed Shear Walls Subjected to Reversed Cyclic Lateral Loading*, International Council for Building Research Studies and Documentation, Ahus, Sweden.

C9. Seismic Isolation and Energy Dissipation (Systematic Rehabilitation)

C9.1 Introduction

Seismic isolation and energy dissipation systems are viable design strategies for seismic rehabilitation of buildings. Other special seismic systems—including active control, hybrid combinations of active and passive energy devices, tuned mass and liquid dampers—are being developed and may provide practical solutions in the near future. These systems include devices that enhance building performance primarily by modifying building response characteristics.

Conceptually, isolation reduces response of the superstructure by “decoupling” the building from the ground. Typical isolation systems reduce forces transmitted to the superstructure by lengthening the period of the building and adding some amount of damping. Added damping is an inherent property of most isolators, but may also be provided by supplemental energy dissipation devices installed across the isolation interface. Under favorable conditions, the isolation system reduces drift in the superstructure by a factor of at least two—and sometimes by as much as factor of five—from that which would occur if the building were not isolated. Accelerations are also reduced in the structure, although the amount of reduction depends on the force-deflection characteristics of the isolators and may not be as significant as the reduction of drift. Reduction of drift in the superstructure protects structural components and elements, as well as nonstructural components sensitive to drift-induced damage. Reduction of acceleration protects nonstructural components that are sensitive to acceleration-induced damage.

Passive energy dissipation devices add damping (and sometimes stiffness) to the building’s structure. A wide variety of passive energy dissipation devices are available, including fluid viscous dampers, viscoelastic materials, and hysteretic devices. Ideally, energy dissipation devices dampen earthquake excitation of the structure that would otherwise cause higher levels of response, and damage to components and elements of the building. Under favorable conditions, energy dissipation devices reduce drift of the structure by a factor of about two to three, if no stiffness is added, and by larger factors if the devices also add stiffness to 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.

Active control systems sense and resist building motion, either by applying external force or by modifying structural properties of active elements (e.g., so-called “smart” braces). Tuned mass or liquid dampers modify properties and add damping to key building modes of vibration. There are other types of special seismic systems, and additional concepts will be undoubtedly be developed in the future.

Consideration of special seismic systems, such as isolation or energy dissipation systems, should be made early in the design process and be based on the Rehabilitation Objectives established for the building (Chapter 2). Whether a special seismic system is found to be the “correct” design strategy for building rehabilitation will depend primarily on the performance required at the specified level of earthquake demand. In general, special seismic systems will be found to be more attractive as a rehabilitation strategy for buildings that have more stringent Rehabilitation Objectives (i.e., higher levels of performance and more severe levels of earthquake demand). Table C9-1 provides some simple guidance on the Performance Levels for which isolation and energy dissipation systems should be considered as possible design strategies for building rehabilitation.

Table C9-1 Applicability of Isolation and Energy Dissipation Systems

Performance Level	Performance Range	Isolation	Energy Dissipation
Operational	Damage Control	Very Likely	Limited
Immediate Occupancy		Likely	Likely
Life Safety	Limited Safety	Limited	Likely
Collapse Prevention		Not Practical	Limited

Table C9-1 suggests that isolation systems should be considered for achieving the Immediate Occupancy Structural Performance Level and the Operational Nonstructural Performance Level. Conversely, isolation will likely not be an appropriate design strategy for achieving the Collapse Prevention Performance Level. In general, isolation systems provide significant protection to the building structure, nonstructural components, and contents, but at a cost that precludes practical application when the budget and Rehabilitation Objectives are modest.

Energy dissipation systems should be considered in a somewhat broader context than isolation systems. For the taller buildings (where isolation systems may not be feasible), energy dissipation systems should be considered as a design strategy when performance goals include the Damage Control Performance Range. Conversely, certain energy dissipation devices are quite economical and might be practical for performance goals that address only Limited Safety. In general, however, energy dissipation systems are more likely to be an appropriate design strategy when the desired Performance Level is Life Safety, or perhaps Immediate Occupancy. 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.

C9.2 Seismic Isolation Systems

Section C9.2.1 of this *Commentary* provides background on seismic isolation concepts and the development, approach, and philosophy of pertinent design codes including the seismic isolation provisions of the 1994 *NEHRP Recommended Provisions for Seismic Regulations for New Buildings* (BSSC, 1995). Section 2.6 (Provisions for Seismically Isolated Structures) of the 1994 *NEHRP Provisions* (plus changes proposed for the 1997 edition of these provisions) is the primary basis and reference for the isolation system design criteria of Section 9.2 of these *Guidelines*.

Section C9.2.1 also provides background on projects in the United States that have utilized isolation as a design strategy for seismic rehabilitation. Motivating factors for selecting isolation are discussed, and guidance is provided for establishing objectives and design criteria appropriate for the desired Performance Level.

Section C9.2.2 describes in detail the mechanical properties and modeling theory for various types of isolation devices. This information is intended as reference material for *Guidelines* users who are interested in better understanding the characteristics and behavior of isolators, or who need to develop detailed mathematical models of isolation system components.

Section C9.2.3 provides comment on the selection of design criteria for seismic isolation, in particular the selection of an appropriate linear or nonlinear procedure. Sections C9.2.4 and C9.2.5 discuss linear and nonlinear procedures, respectively, focusing on methods that are unique to isolation.

Commentary is not provided for Sections 9.2.6 (Nonstructural Components), 9.2.7 (Detailed System Requirements), 9.2.8 (Design and Construction Review), and 9.2.9 (Isolation System Testing and Design Properties) of the *Guidelines.* These sections are similar in content to corresponding sections of the 1994 *NEHRP Provisions* and the 1996 edition of *Recommended Lateral Force Requirements and Commentary*—commonly referred to as the *Blue Book*—produced by the Structural Engineers Association of California (SEAOC, 1996). The reader is directed to the commentaries of these references for discussion of topics not covered in this *Commentary.*

C9.2.1 Background

C9.2.1.1 Development of Isolation Provisions for New Buildings

Until the early 1980s, the design concept of seismic isolation had not been utilized in the United States. As isolation system products matured and became commercially available, research projects led to practice, and isolation began to be seriously considered, particularly for those projects seeking improved seismic performance. This activity identified a need to supplement existing codes with design requirements developed specifically for isolated structures. This need was shared by the public and its agents (i.e., building officials), who required assurance that this new technology was being implemented properly, as well as by the engineering profession, which required a minimum standard for design and construction.

Early efforts directed at creating design provisions for isolated structures began with the Northern Section of SEAOC in the mid-1980s. In 1986, this section of SEAOC published *Tentative Seismic Isolation Design*

Requirements (SEAOC, 1986), the first collection of design provisions for base-isolated structures. These provisions were based on the same seismic criteria required for design of fixed-base buildings, and used similar design concepts, such as the prescription of minimum design force and displacement by formula.

Recognizing the need for a document that would better represent a consensus opinion of all sections of SEAOC, the SEAOC Seismology Committee developed design provisions, "General Requirements for the Design and Construction of Seismic-Isolated Structures," that were published as Appendix 1L of the *1990 SEAOC Blue Book* (SEAOC, 1990). These provisions were also adopted (with minor editorial changes) by the International Conference of Building Officials (ICBO) and published as a nonmandatory appendix to Chapter 23 of the 1991 *Uniform Building Code* (UBC) (ICBO, 1991). The Seismology Committee of SEAOC and ICBO have revised their respective design provisions periodically, and current versions of isolation system criteria may be found in the 1996 *SEAOC Blue Book* (SEAOC, 1996) or the 1994 *UBC* (ICBO, 1994).

In 1992, Technical Subcommittee 12 (TS-12) of the 1994 Provisions Update Committee was formed by the Building Seismic Safety Council to incorporate design requirements for base isolation and energy dissipation systems into the 1994 *NEHRP Recommended Provisions*. TS-12 based its recommendations directly on the isolation provisions of the 1994 *UBC*, modified to conform to the strength-design approach and nomenclature of the *Provisions*. In general, the design provisions for isolated buildings found in Section 2.5 of the *Provisions* conform to those of the *UBC*. Differences between the *Provisions* and the *UBC* will be resolved in the 1997 editions of these documents, when both sets of provisions are based on strength design.

The 1994 *NEHRP Recommended Provisions* and the changes proposed by TS-12 for the 1997 *NEHRP Recommended Provisions* for new buildings were used as resource documents for the development of the *Guidelines* for seismic isolation rehabilitation of existing buildings. The following section of the *Commentary* discusses the philosophy and criteria underlying the NEHRP/UBC/SEAOC provisions for seismic isolation of new buildings (Kircher and Bachman, 1991).

C9.2.1.2 Design Philosophy for Isolation Provisions for New Buildings

The underlying philosophy guiding the development of the NEHRP/UBC/SEAOC provisions for isolation of new buildings may be characterized as a combination of the primary performance objective for fixed-base buildings—which is the protection of life safety for a major earthquake—and the additional performance objective of damage reduction, an inherent benefit of seismic isolation. The design criteria of the NEHRP/UBC/SEAOC provisions are based on a combination of life safety and damage reduction goals. These criteria are summarized in the following statements.

1. The NEHRP/UBC/SEAOC provisions specify two levels of earthquake: the BSE-1 (referred to as the Design Basis Earthquake in SEAOC/UBC provisions) and the Maximum Capable Earthquake.

   The BSE-1 is the same earthquake level of ground shaking as that required by the NEHRP/UBC/SEAOC provisions for design of fixed-base structures: a level of ground motion that has a 10% probability of being exceeded in a 50-year time period (BSE-1 earthquake).

   In this Chapter 9, the design earthquake filling this role for the rehabilitation of existing buildings is user-specified.

   The Maximum Capable Earthquake is an additional, higher level of earthquake ground motion defined as the maximum level of ground shaking that may be expected at the building site within the known geological framework. The 1994 editions of the NEHRP/UBC/SEAOC provisions permit this level to be taken as the level of earthquake ground motion that has a 10% probability of being exceeded in a 100-year time period (10%/100 year earthquake).

   In this Chapter 9, the Maximum Considered Earthquake fills this role for the rehabilitation of existing buildings.

2. The NEHRP/UBC/SEAOC provisions for new buildings require the isolation system to be capable of sustaining loads corresponding to the Maximum Capable Earthquake without failure (e.g., the isolation system is to be designed and tested for Maximum Capable Earthquake displacement). Likewise, the provisions require building separations and utilities that cross the isolation

interface to be designed to accommodate Maximum Capable Earthquake displacement.

3. The NEHRP/UBC/SEAOC provisions require the structure (above the isolation system) to remain “essentially elastic” for the design earthquake, which may be specified as the BSE-1 (e.g., inelastic response of the lateral-load-resisting superstructure system is limited to about one-third of that permitted by the NEHRP/UBC/SEAOC provisions for design of a comparable, fixed-base building).

Design provisions for fixed-base buildings provide reasonable protection against major structural failure and loss of life, but are not intended “to limit damage, maintain functions, or provide for easy repair” (SEAOC, 1996). Based on this philosophy, the lateral forces required for strength design of fixed-base structures are as little as one-eighth of the force level that would occur in buildings responding elastically during a major earthquake, if the structure remained fully elastic. Life safety is provided by design provisions that require the structural system to have sufficient ductility and stability to displace significantly beyond the elastic limit without gross failure or collapse. However, damage to structural elements, nonstructural components, and/or contents of a fixed-base building can occur during an earthquake and would be likely for a major event.

The NEHRP/UBC/SEAOC provisions for fixed-base buildings are based on earthquake forces corresponding to the BSE-1 (reduced for design of elements, as discussed above). Survival for response beyond the BSE-1 level is implicitly addressed by special ductility and detailing requirements. In contrast, the NEHRP/UBC/SEAOC provisions for isolated buildings explicitly consider response beyond the design earthquake or the BSE-1 by requiring the isolation system to be designed for displacements corresponding to the Maximum Capable Earthquake, an event that represents “worst-case” earthquake demands on the isolation system. The intent of requiring the isolation system to be explicitly designed (and verified) for Maximum Capable Earthquake displacement is to provide reasonable assurance that the isolation system will be at least as “safe” as a fixed-base structure. Explicit design (and testing) of the isolation system for “worst-case” earthquake displacement is necessary at this time because a sufficient base of experience does not exist that would justify less conservative criteria.

Ideally, lateral displacement of an isolated structure occurs in the isolation system, rather than in the superstructure above. The lateral-load-resisting system of the superstructure should be designed to have sufficient stiffness and strength to avoid large inelastic displacements. For this reason, the NEHRP/UBC/SEAOC provisions contain criteria that limit the inelastic response of the superstructure to a fraction of that permitted for a fixed-based building. Although damage control for the design earthquake or the BSE-1 is not an explicit objective of the NEHRP/UBC/SEAOC provisions, an isolated structure designed for limited inelastic response of the superstructure will also reduce the level of damage that would otherwise occur during an earthquake. Isolated structures designed in conformance with the NEHRP/UBC/SEAOC provisions should, in general, be able to:

1. Resist minor and moderate levels of earthquake ground motion without damage to structural elements, nonstructural components, or building contents

2. Resist major levels of earthquake ground motion without any of the following occurring: (a) failure of the isolation system, (b) significant damage to structural elements, (c) extensive damage to nonstructural components, or (d) major disruption to facility function

The performance objectives for isolated structures, stated above, considerably exceed the performance anticipated for fixed-base structures during moderate and major earthquakes. Table C9-2 provides a tabular comparison of the performance expected for isolated and fixed-base structures designed in accordance with NEHRP/UBC/SEAOC provisions. Loss of function is not included in this table. For certain (fixed-base) facilities, loss of function would not be expected to occur until there is significant structural damage causing closure of, or restricted access to the building. In other cases, the facility could have only limited or no structural damage, but would not be functional as a result of damage to vital nonstructural components and contents. Isolation would be expected to mitigate structural and nonstructural damage, and to protect the facility against loss of function.

C9.2.1.3 Overview of Seismic Isolation Rehabilitation Projects

A number of buildings have been (or are in the process of being) rehabilitated using seismic isolation. These

Table C9-2 Protection Intended for New Buildings

Risk Category	Earthquake Ground Motion Level		
	Minor	Moderate	Major
Life Safety[1]	F/I	F/I	F/I
Structural Damage[2]	F/I	F/I	I
Nonstructural Damage[3] (Contents Damage)	F/I	I	I

1. Loss of life is not expected for fixed-base (F) or isolated (I) buildings.
2. Significant structural damage is not expected for fixed-base (F) or isolated (I) buildings.
3. Significant nonstructural (contents) damage is not expected for fixed-base (F) or isolated (I) buildings

buildings include the Salt Lake City and County Building in Salt Lake City, Utah (Mayes, 1988), the Rockwell Building in Seal Beach, California (Hart et al., 1990), the Hawley Apartments in San Francisco, California (Zayas and Low, 1991), the Mackay School of Mines in Reno, Nevada (Way and Howard, 1990), the U.S. Court of Appeals, San Francisco, California (Amin et al., 1993), Oakland City Hall in Oakland, California (Honeck et al., 1993), and San Francisco City Hall (Naaseh, 1995). A summary of these projects is provided in Table C9-3.

The rehabilitation projects summarized in Table C9-3 range in size from a 20,000-square-foot building to buildings of up to 500,000 square feet. The original structural systems of these buildings include wood bearing walls, nonductile reinforced concrete moment frames, and steel moment frames with unreinforced masonry (URM) infill and URM bearing walls. Most of the buildings are owned by a local, state, or federal government agency and often have historical significance. The collective size of the buildings in Table C9-3 is over 3 million square feet, and their combined value is close to $1 billion.

The types of isolators used to date in the United States to rehabilitate buildings include lead-rubber bearing (LRB) isolators, rubber-bearing (RB) isolators, friction-pendulum system (FPS) isolators, high-damping rubber bearing (HDR) isolators, and sliding polytetrafluoroethylene (PTFE) isolators. These five types of isolators are representative of the range of products currently available in the US. The projects listed in Table C9-3 have required as few as 31 isolators for the Hawley Apartments, a four-story, 20,000-square-foot residential building, and as many as 591 isolators for San Francisco City Hall, a five-story, 500,000-square-foot historical structure. The extent of new structure added above the isolation system also varies greatly from one project to another. In some cases, such as the Mackay School of Mines, only minimal strengthening of the original structure was required. In other cases, such as the Rockwell Building, the superstructure was substantially strengthened by the addition of new framing at the building perimeter.

C9.2.1.4 Seismic Isolation Rehabilitation Goals

The philosophy or purpose for seismic rehabilitation using isolation is directly dependent on the owner's motivation to upgrade the building, and expectations of upgraded building performance during and following an earthquake. For this reason, Rehabilitation Objectives may vary greatly from project to project.

To date, there are five primary considerations, listed and described below, that have motivated owners to choose isolation for rehabilitation of existing buildings. With each consideration, one or more project(s) are identified that selected seismic isolation for building rehabilitation based on that consideration as well as others.

1. **Functionality**. The facility should remain open and operational during and after an earthquake or be able to resume operation within a short period of time (e.g., Rockwell Building, computer/financial center operation).

2. **Contents Protection**. Important contents must be protected against damage due to earthquake shaking (e.g., San Francisco Asian Art Museum, $3 billion of art contents).

3. **Investment Protection**. Long-term economic loss due to earthquake damage should be mitigated (e.g., State of California Justice Building; Pyle et al., 1993).

4. **Historical Building Preservation**. Seismic rehabilitation modification or demolition of historical building features must be minimized (e.g., Salt Lake City and County Building, Oakland City Hall, U.S. Court of Appeals, and San Francisco City Hall).

Table C9-3 Summary of US Seismic Isolation Rehabilitation Projects

Building/Project Information			***Structural Information***		
Name (Location)	**Status**	**Size in Sq. Ft.**	**Isolation System**	**Original Structure**	**New Structure**
Salt Lake City and County Building (Salt Lake City, UT)	Complete (1988)	170,000	447 Isolators (208 LRB + 239 RB + PTFE)	1894 5-story URM bearing wall w/clock tower (240' total height)	Steel braced frame (clock tower only)
Rockwell Building (Seal Beach, CA)	Complete (1991)	300,000	78+ Isolators (52 LRB + 26 RB + PTFE)	1967 8-story RC moment frame	RC moment frame at perimeter, floors 1–6
Hawley Apartments (San Francisco, CA)	Complete (1991)	20,000	31 Isolators (FPS)	1920 4-story wood bearing wall	Steel moment frame at first floor
Mackay School of Mines (Reno, NV)	Complete (1993)	50,000	106 Isolators (64 HDR + 42 PTFE)	1908 3-story URM bearing wall	Floor ties/wall anchors (new basement)
Campbell Hall, Western Oregon State College (Monmouth, OR)	Complete (1994)		42+ Isolators (26 LRB + 16 RB + PTFE)	1872–1898 3-story URM bearing wall	
Oakland City Hall (Oakland, CA)	Complete (1995)	153,000	126 Isolators (42 LRB + 69 RB + 15 PTFE)	1914 18-story steel frame/ URM in-fill w/clock tower (324' total height)	RC shear walls at cores, steel braced frame at clock tower
U.S. Court of Appeals (San Francisco, CA)	Complete (1995)	350,000	256 Isolators (FPS)	1905 4-story steel frame/URM in-fill with 1933 addition	RC shear walls
Long Beach Veterans Admin. Hospital (Long Beach, CA)	Complete (1995)	350,000	156 Isolators (110 LRB + 18 RB + 30 PTFE)	1967 12-story RC perforated shear wall	Basement columns strengthened
Building S-12 Hughes (El Segundo, CA)	Complete (1995)	240,000	45+ Isolators (24 LRB + 21 RB + PTFE)	1960s 12-story RC shear wall/frame building	First floor and substructure strengthened
Kerckhoff Hall, Univ. of California, Los Angeles (Westwood, CA)	Complete (1996)	92,000	126+ Isolators (33 LRB + 93 RB + PTFE)	6-story RC and brick wall structure	First floor and substructure strengthened
San Francisco City Hall (San Francisco, CA)	Complete (1997)	500,000	591 Isolators (530 LRB + 61 PTFE)	1912 5-story steel frame/URM in-fill with dome (~300' total height)	Steel braced frame in dome and RC shear walls at lower floors

LRB: Lead-rubber bearing isolators

RB: Rubber bearing isolators

PTFE: Sliding polytetra fluoroethylene isolators

FPS: Friction pendulum system isolators

HDR: High damping rubber bearing isolators

5. **Construction Economy**. The building is of a size and/or complexity that makes seismic isolation the most economical construction alternative (e.g., Oakland and San Francisco City Halls).

Each rehabilitation project will have a different set of motivating factors and related performance objectives, and therefore will likely require different design criteria. The first essential step in developing design criteria is to identify and rank the owner's seismic risk goals in terms of facility function, damage and investment protection, historical preservation, and construction economy. These goals will guide the engineer's selection of performance objectives and design criteria appropriate for the building. Owners who place a high priority on functionality or protection of contents or investment will require more stringent design criteria, such as those in the *Guidelines* for Immediate Occupancy. Owners more intent on historical preservation or construction economy will require less stringent design criteria, such as those in the *Guidelines* for Life Safety. Owners that are only interested in Collapse Prevention should probably consider other, more economical design strategies than seismic isolation.

C9.2.2 Mechanical Properties and Modeling of Seismic Isolation Systems

C9.2.2.1 General

The three basic properties of an isolation system are: (1) horizontal flexibility to increase structural period and reduce spectral demands (except for very soft soil sites), (2) energy dissipation (also known as damping) to reduce displacements, and (3) sufficient stiffness at small displacements to provide adequate rigidity for service-level environmental loadings. The horizontal flexibility common to all practical isolation systems serves to uncouple the building from the effects of high-frequency earthquake shaking typical of rock or firm soil sites—thus serving to deflect the earthquake energy and significantly reduce the magnitude of the resulting inertia forces in the building. Energy dissipation in an isolation system, in the form of either hysteretic or viscous damping, serves to reduce the displacement response of an isolation system (Skinner et al., 1993; Kelly, 1993; Soong and Constantinou, 1994), generally resulting in more compact isolators.

The reduction of bearing displacements in highly damped isolation systems typically results in reduction of the shear force in the isolation system. This is demonstrated in Figures C9-1 and C9-2. The results are from nonlinear time history analyses of an eight-story isolated building supported by 45 isolators (Winters and Constantinou, 1993; Soong and Constantinou, 1994). Each isolator has bilinear hysteretic properties that characterize a wide range of elastomeric and sliding isolation systems. A total of twelve isolation systems, having an isolated period (T_I) in the range of 1.5 to 3 seconds and effective damping (β_{eff}) in the range of 0.06 to 0.37, were analyzed. The seismic input was representative of Seismic Zone 4, Soil Profile Type S_2, of the 1991 *UBC* (ICBO, 1991). This input consisted of nine pairs of earthquakes, with each pair applied along the principal directions of the structure.

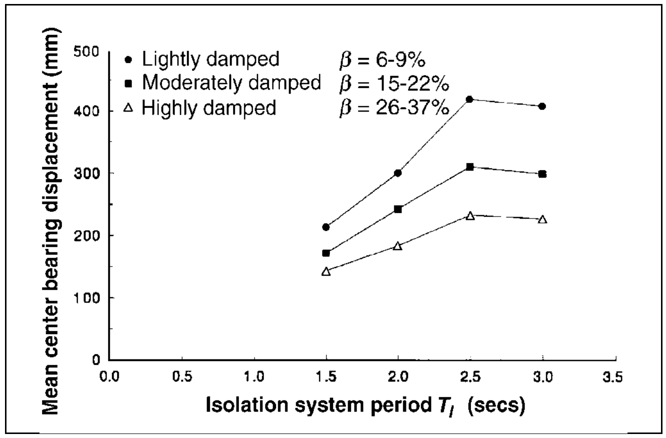


Figure C9-1 Center Bearing Displacement (Mean of Nine Analyses) in Eight-Story Building with Hysteretic Isolation System

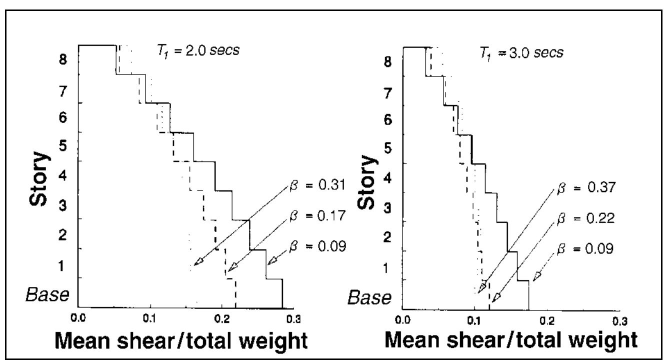


Figure C9-2 Distribution of Shear Force (Mean of Nine Analyses) with Height in Eight-Story Building with Hysteretic Isolation System

Figure C9-1 demonstrates the increase of bearing displacement with (1) increasing period, and (2) decreasing effective damping. Figure C9-2 demonstrates the reduction of shear force in the isolation system (termed "base" in the figure) with increasing effective damping. Note in this figure that for highly damped isolation systems, the shear force distribution is nearly constant over the height of the structure, whereas for lightly damped systems this distribution is approximately triangular. The latter is indicative of response in the fundamental mode of vibration, whereas the former is indicative of higher mode response, which is typically accompanied by higher accelerations in upper floors. Nevertheless, the benefits offered by highly damped systems are evident. For example, in the system with an isolated period equal to 2.0 seconds, an effective damping of 0.31 results in a 40% reduction in bearing displacements and lower structural shear forces in the bottom two-thirds of the structure, all in comparison with the response of a lightly damped (β_{eff} = 0.09) system. However, the accelerations in the top floor of the building with the highly damped isolators are 40% higher than those in the lightly damped building. Thus, highly damped systems offer advantages when the primary intent of seismic isolation is to protect the structural system. Lightly damped systems may be preferable when the intent of seismic isolation is to protect secondary systems, such as sensitive equipment (Kelly, 1993; Skinner et al., 1993). Typical seismic isolation systems are horizontally flexible and vertically stiff. Vertical ground motions are likely to be amplified in most isolation systems. If protection of secondary systems is of primary importance, due consideration of vertical ground motion is necessary; vertical isolation of either the building or individual secondary systems may also be appropriate.

The benefits of reduced bearing displacements, shear forces, and accelerations may be realized with linear seismic isolation systems. For example, Figure C9-3 compares the distribution of shear force over the height of an eight-story building for highly damped isolation systems that have either bilinear hysteretic behavior, or linearly elastic and linearly viscous behavior. A system consisting of low-damping elastomeric bearings and linear fluid viscous devices has substantially linear behavior and offers the benefits of reduced bearing displacements, shear forces, and floor accelerations. Skinner et al. (1993) provide several examples that demonstrate many of these features of seismic isolation for a wide range of isolation system properties.

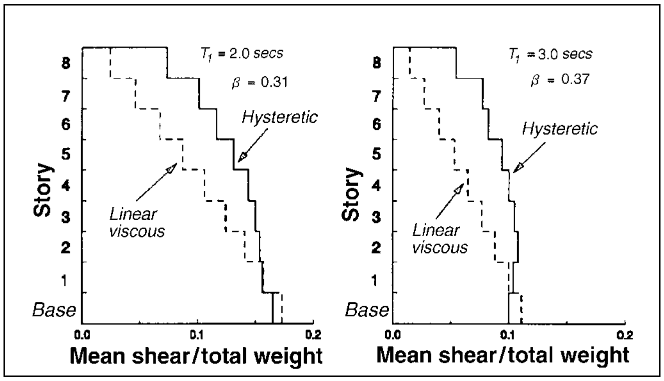


Figure C9-3 Comparison of Distribution of Shear Force with Height in Eight-Story Building with Hysteretic and Linear Viscous Isolation System

C9.2.2.2 Mechanical Properties of Seismic Isolators

A. Elastomeric Isolators

Elastomeric bearings represent a common means for introducing flexibility into an isolated structure. They consist of thin layers of natural rubber that are vulcanized and bonded to steel plates. Natural rubber exhibits a complex mechanical behavior, which can be described simply as a combination of viscoelastic and hysteretic behavior. Low-damping natural rubber bearings exhibit essentially linearly elastic and linearly viscous behavior at large shear strains. The effective damping is typically less than or 0.07 for shear strains in the range of 0 to 2.0.

Lead-rubber bearings are generally constructed of low-damping natural rubber with a preformed central hole, into which a lead core is press-fitted. Under lateral deformation, the lead core deforms in almost pure shear, yields at low level of stress (approximately 8 to 10 MPa in shear at normal temperature), and produces hysteretic behavior that is stable over many cycles. Unlike mild steel, lead recrystallizes at normal temperature (about 20°C), so that repeated yielding does not cause fatigue failure. Lead-rubber bearings generally exhibit characteristic strength that ensures rigidity under service loads. Figure C9-4 shows an idealized force-displacement relation of a lead-rubber bearing. The characteristic strength, Q, is related to the lead plug area, A_p, and the shear yield stress of lead, σ_{YL}:

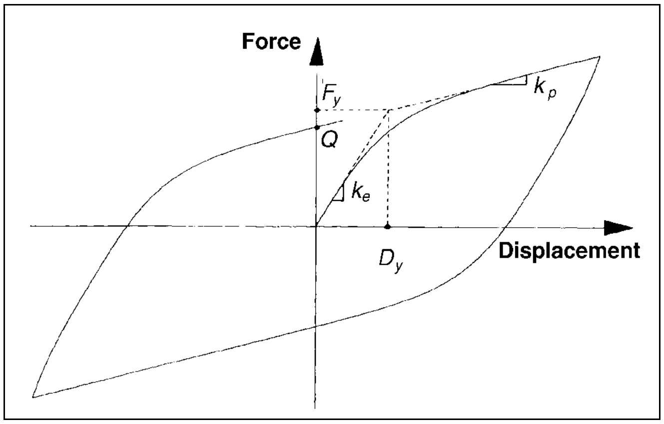


Figure C9-4 Idealized Hysteretic Force-Displacement Relation of Elastomeric Bearing

$$Q = A_p \sigma_{YL} \quad \text{(C9-1)}$$

The post-yield stiffness, k_p, is typically higher than the shear stiffness of the bearing without the lead core:

$$k_p = \frac{A_r G f_L}{\Sigma t} \quad \text{(C9-2)}$$

where A_r is the bonded rubber area, Σt is the total rubber thickness, G is the shear modulus of rubber (typically computed at shear strain of 0.5), and f_L is a factor larger than unity. Typically, f_L is 1.15, and the elastic stiffness ranges between 6.5 to 10 times the post-yield stiffness.

The behavior of lead-rubber bearings may be represented by a bilinear hysteretic model. Computer programs *3D-BASIS* (Nagarajaiah et al., 1991; Reinhorn et al., 1994; Tsopelas et al., 1994) and *ETABS, Version 6* (CSI, 1994) have the capability of modeling hysteretic behavior for isolators. These models typically require definition of three parameters, namely, the post-yield stiffness k_p, the yield force F_y, and the yield displacement D_y. For lead-rubber bearings in which the elastic stiffness is approximately equal to 6.5 k_p, the yield displacement can be estimated as:

$$D_y = \frac{Q}{5.5 k_p} \quad \text{(C9-3)}$$

The yield force is then given by

$$F_y = Q + k_p D_y \quad \text{(C9-4)}$$

High-damping rubber bearings are made of specially compounded rubber that exhibits effective damping between 0.10 and 0.20 of critical. The increase in effective damping of high-damping rubber is achieved by the addition of chemical compounds that may also affect other mechanical properties of rubber.
Figure C9-5 shows representative force-displacement loops of a high-damping rubber bearing under scragged conditions.

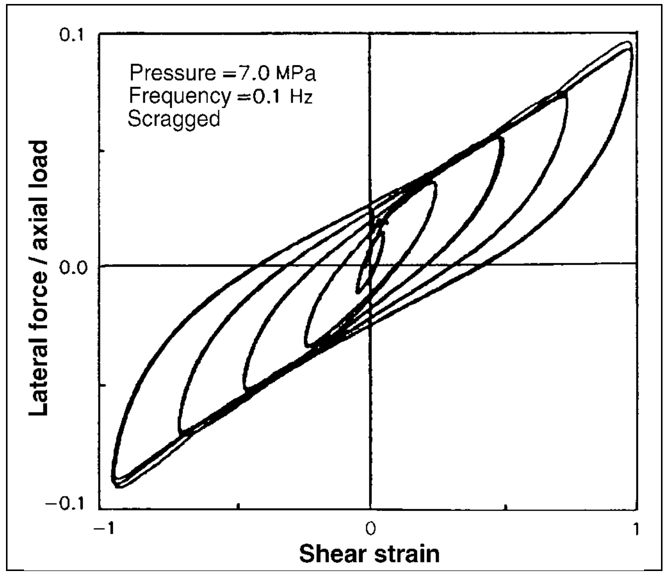


Figure C9-5 Force-Displacement Loops of a High-Damping Rubber Bearing

Scragging is the process of subjecting an elastomeric bearing to one or more cycles of large amplitude displacement. The scragging process modifies the molecular structure of the elastomer and results in more stable hysteresis at strain levels lower than that to which the elastomer was scragged. Although it is usually assumed that the scragged properties of an elastomer remain unchanged with time, recent studies (Cho and Retamal, 1993; Murota et al., 1994) suggest that partial recovery of unscragged properties is likely. The extent of this recovery is dependent on the elastomer compound.

Mathematical models capable of describing the transition between virgin and scragged properties of high-damping rubber bearings are not yet available. It is appropriate in this case to perform multiple analyses with stable hysteretic models and obtain bounds on the dynamic response. A smooth bilinear hysteretic model that is capable of modeling the behavior depicted in Figure C9-4 is appropriate for such analyses, as long as the peak shear strain is below the stiffening limit of approximately 1.5 to 2.0, depending on the rubber compound. Beyond this strain limit many elastomers exhibit stiffening behavior, with tangent stiffness approximately equal to twice the tangent stiffness prior to initiation of stiffening. For additional information, refer to Tsopelas et al. (1994).

To illustrate the calculations of parameters from prototype bearings test data, Figure C9-6 shows experimentally determined properties of the high-damping rubber bearings, for which loops are shown in Figure C9-5. The properties identified are the tangent shear modulus, G, and the effective damping ratio, β_{eff} (described by Equation C9-18, which is now defined for a single bearing rather than the entire isolation system) under scragged conditions. With reference to Figure C9-4, G is related to the post-yielding stiffness k_p.

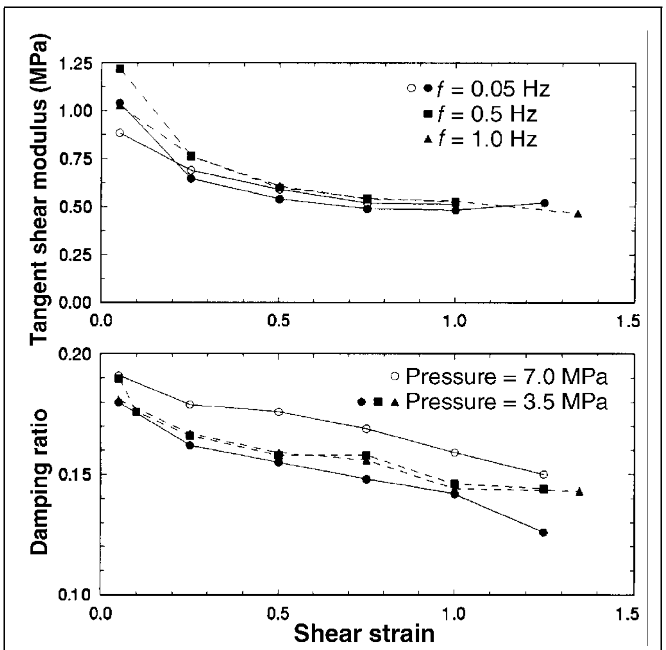


Figure C9-6 ***Tangent Shear Modulus and Effective Damping Ratio of High-Damping Rubber Bearing***

$$k_p = \frac{GA}{\Sigma t} \qquad \text{(C9-5)}$$

where A is the bonded rubber area. The results of Figure C9-6 demonstrate that the tangent shear modulus and equivalent damping ratio are only marginally affected by the frequency of loading and the bearing pressure, within the indicated range for the tested elastomer. Different conclusions may be drawn from testing of other high-damping rubber compounds.

The parameters of the bilinear hysteretic model may be determined by use of the mechanical properties G and β_{eff} at a specific shear strain, such as the strain corresponding to the design displacement D. The post-yield stiffness k_p is determined from Equation C9-5, whereas the characteristic strength, Q, can be determined as:

$$Q = \frac{\pi\beta_{eff}k_pD^2}{(2-\pi\beta_{eff})D - 2D_y} \qquad \text{(C9-6)}$$

where D_y is the yield displacement. The yield displacement is generally not known a priori. However, experimental data suggest that D_y is approximately equal to 0.05 to 0.1 times the total rubber thickness, Σt. With the yield displacement approximately determined, the model can be completely defined by determining the yield force (Equation C9-4). It should be noted that the characteristic strength may be alternatively determined from the effective stiffness, k_{eff} (Equation C9-17), of the bearing, as follows:

$$Q = \frac{\pi\beta_{eff}k_{eff}D^2}{2(D-D_y)} \qquad \text{(C9-7)}$$

The effective stiffness is a more readily determined property than the post-yielding stiffness. The effective stiffness is commonly used to obtain the effective shear modulus, G_{eff}, defined as:

$$G_{eff} = \frac{k_{eff}\Sigma t}{A} \qquad \text{(C9-8)}$$

The behavior of the bearing for which the force-displacement loops are shown in Figure C9-5 is now analytically constructed using the mechanical properties at a shear strain of 1.0 and a bearing pressure of 7.0 MPa. These properties are $G_{eff} = 0.50$ MPa and $\beta_{eff} = 0.16$. With the bonded area and total thickness of rubber known, and assuming $D_y = 0.1\Sigma t$, a bilinear hysteretic model was defined and implemented in the program *3D-BASIS*. The simulated loops are shown in Figure C9-7, where it may be observed that the calculated hysteresis loop at shear strain of 1.0 agrees well with the corresponding experimental hysteresis loop. However, at lower peak shear strain the analytical loops have a constant characteristic strength, whereas the experimental loops have a characteristic strength dependent on the shear strain amplitude. Nevertheless, the analytical model will likely produce acceptable results when the design parameters are based on the mechanical properties at a strain corresponding to the design displacement.

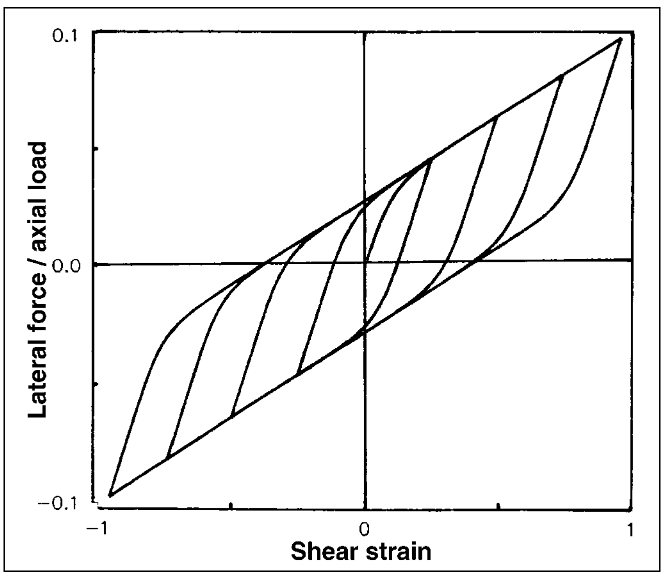


Figure C9-7 Analytical Force-Displacement Loops of High-Damping Rubber Bearing

Elastomeric bearings have finite vertical stiffness that affects the vertical response of the isolated structure. The vertical stiffness of an elastomeric bearing may be obtained from

$$k_v = \frac{E_c A}{\Sigma t} \tag{C9-9}$$

where E_c is the compression modulus. Although a number of approximate empirical relations have been proposed for the calculation of the compression modulus, the correct expression for circular bearings is

$$E_c = \left(\frac{1}{6G_{eff}S^2} + \frac{4}{3K}\right)^{-1} \tag{C9-10}$$

(Kelly, 1993) where K is the bulk modulus (typically assumed to have a value of 2000 MPa) and S is the shape factor, which is defined as the ratio of the loaded area to the bonded perimeter of a single rubber layer. For a circular bearing of bonded diameter ϕ and rubber layer thickness t, the shape factor is given by

$$S = \frac{\phi}{4t} \tag{C9-11}$$

Seismic elastomeric bearings are generally designed with large shape factor, typically 12 to 20. Considering an elastomeric bearing design with $S = 15$, $G_{eff} = 1$ MPa, and $K = 2000$ MPa, the ratio of vertical stiffness (Equation C9-9) to effective horizontal stiffness (Equation C9-8) is approximately equal to 700. Thus, the vertical period of vibration of a structure on elastomeric isolation bearings will be about 26 times (i.e., $\sqrt{700}$) less than the horizontal period; on the order of 0.1 second. This value of vertical period provides potential for amplification of the vertical ground acceleration by the isolation system. The primary effect of this amplification is to change the vertical load on the bearings, which may need to be considered for certain design applications.

Another consideration in the design of seismically isolated structures with elastomeric bearings is reduction in height of a bearing with increasing lateral deformation (Kelly, 1993). While this reduction of height is typically small, it may be of importance when elastomeric bearings are combined with other isolation elements that are vertically rigid (such as sliding bearings). In addition, incompatibilities in vertical displacements may lead to a redistribution of loads.

B. Sliding Isolators

Sliding bearings will tend to limit the transmission of force to an isolated structure to a predetermined level. While this is desirable, the lack of significant restoring force can result in significant variations in the peak displacement response, and can result in permanent offset displacements. To avoid these undesirable features, sliding bearings are typically used in combination with a restoring force mechanism.

The lateral force developed in a sliding bearing can be defined as:

$$F = \frac{N}{R}U + \mu_s N \operatorname{sgn}(\dot{U}) \tag{C9-12}$$

where

U = Displacement
$\dot{U}$ = Sliding velocity
R = Radius of curvature of sliding surface
μ_s = Coefficient of sliding friction
N = Normal load on bearing

The normal load consists of the gravity load, W, the effect of vertical ground acceleration, $\ddot{U}_V$, and the additional seismic load due to overturning moment, P_s:

$$N = W\left(1 + \frac{\ddot{U}_V}{g} + \frac{P_s}{W}\right) \tag{C9-13}$$

The first term in Equation C9-12 denotes the restoring force component, and the second term describes the friction force. For flat sliding bearings the radius of curvature is infinite, so that the restoring force term in Equation C9-12 vanishes. For a spherical sliding surface (Zayas et al., 1987) the radius of curvature is constant, so that the bearing exhibits a linear restoring force; that is, under constant gravity load the stiffness is equal to W/R_o, where R_o is the radius of the spherical sliding surface. When the sliding surface takes a conical shape, the restoring force is constant. Figure C9-8 shows idealized force-displacement loops of sliding bearings with flat, spherical, and conical surfaces.

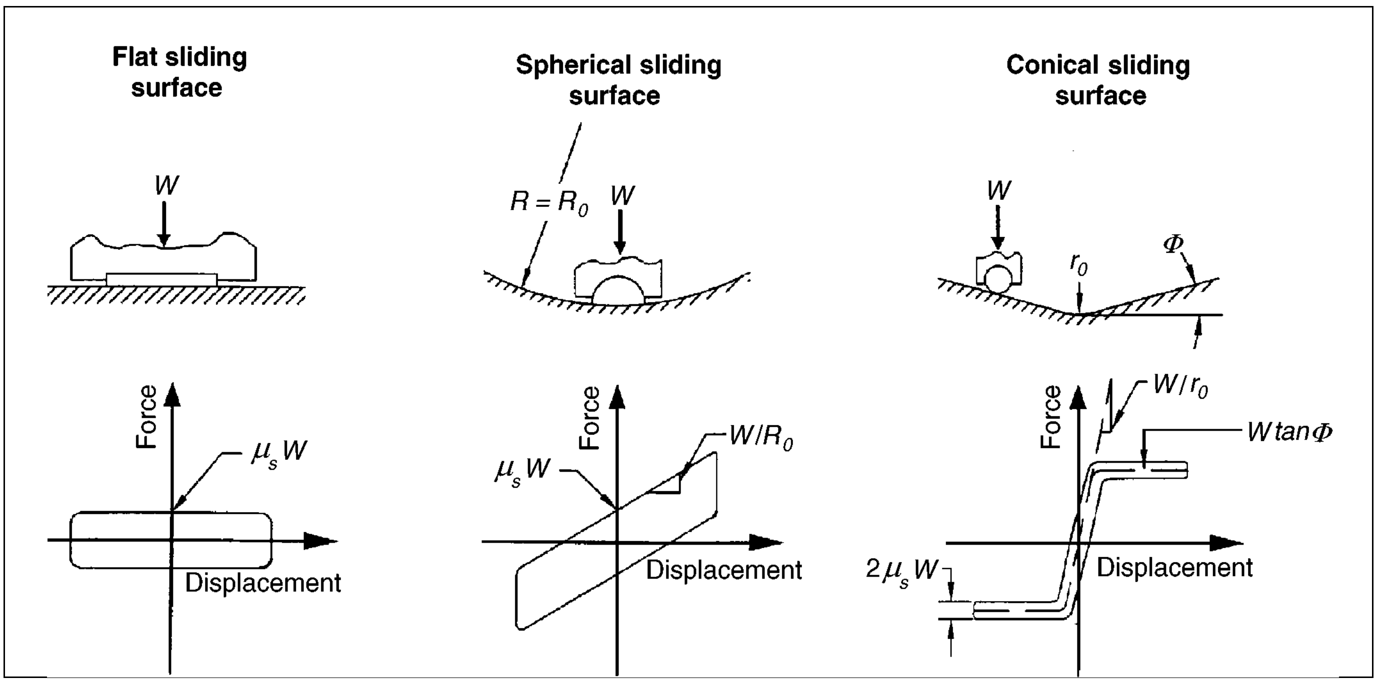


Figure C9-8 Idealized Force-Displacement Loops of Sliding Bearings

Sliding bearings with either a flat or single curvature spherical sliding surface are typically made of PTFE or PTFE-based composites in contact with polished stainless steel. The shape of the sliding surface allows large contact areas that, depending on the materials used, are loaded to average bearing pressures in the range of 7 to 70 MPa. For interfaces with shapes other than flat or spherical, the load needs to be transferred through a bearing as illustrated in Figure C9-8 for the conical sliding surface. Such an arrangement typically results in a very low coefficient of friction.

For bearings with large contact area, and in the absence of liquid lubricants, the coefficient of friction depends on a number of parameters, of which the three most important are the composition of the sliding interface, bearing pressure, and velocity of sliding. For interfaces composed of polished stainless steel in contact with PTFE or PTFE-based composites, the coefficient of sliding friction may be described by

$$\mu_s = f_{max} - (f_{max} - f_{min}) \exp(-a|\dot{U}|) \qquad \text{(C9-14)}$$

where parameters f_{min} and f_{max} describe the coefficient of friction at small and large velocities of sliding and under constant pressure, respectively, all as depicted in Figure C9-9. Parameters f_{max} , f_{min} , and a depend on the bearing pressure, although only the dependency of f_{max} on pressure is of practical significance. A good approximation to the experimental data (Constantinou et al., 1993b) is

$$f_{max} = f_{maxo} - (f_{maxo} - f_{maxp}) \tanh \varepsilon p \qquad \text{(C9-15)}$$

where the physical significance of parameters f_{maxo} and f_{maxp} is as illustrated in Figure C9-9. The term p is the instantaneous bearing pressure, which is equal to the normal load N (Equation C9-13) divided by the contact area; and ε is a parameter that controls the variation of f_{max} with pressure.

Figure C9-9 illustrates another feature of sliding bearings. On initiation of motion, the coefficient of friction exhibits a static or breakaway value, μ_B, which is typically higher than the minimum value f_{min} . To demonstrate frictional properties, Figure C9-10 shows the relation between bearing pressure and the friction coefficients f_{max} , μ_B, and f_{min} of a PTFE-based composite material in contact with polished stainless steel at normal temperature. These data were compiled from testing of bearings in four different testing programs (Soong and Constantinou, 1994).

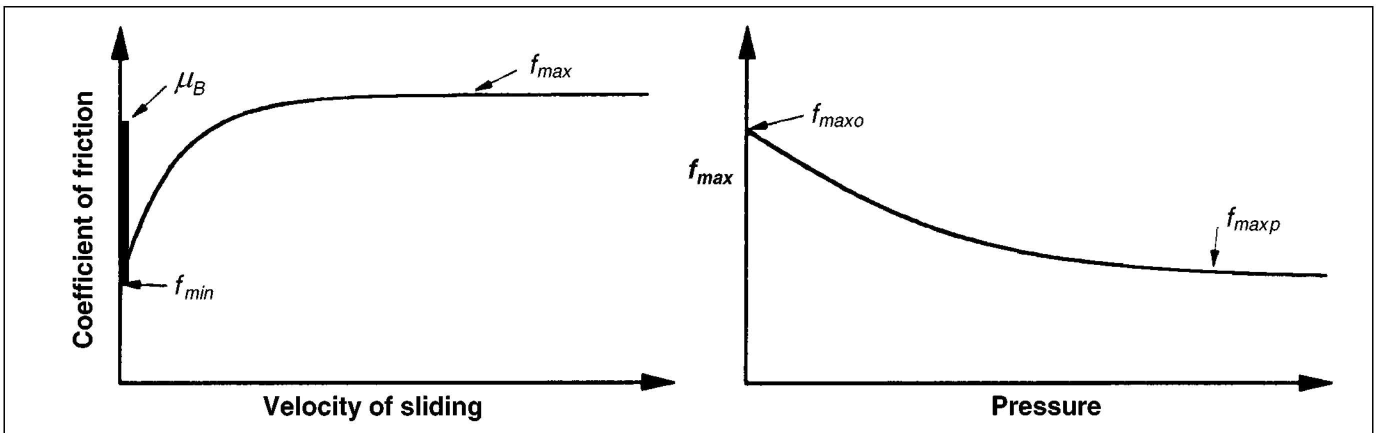


Figure C9-9 Parameters in Model of Friction of Sliding Bearings

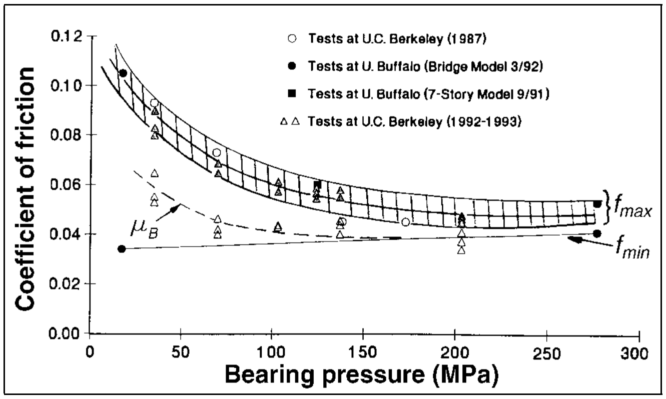


Figure C9-10 Coefficient of Friction of PTFE-based Composite in Contact with Polished Stainless Steel at Normal Temperature

C. Hybrid Isolators

Combined elastomeric-sliding isolation systems have been used in buildings in the United States. Japanese engineers have also used elastomeric bearings in combination with mild steel elements that are designed to yield in strong earthquakes and enhance the energy dissipation capability of the isolation system (Kelly, 1988). These mild steel elements exhibit either elasto-plastic behavior or bilinear hysteretic behavior with low post-yielding stiffness. Moreover, fluid viscous energy dissipation devices have been used in combination with elastomeric bearings. The behavior of fluid viscous devices is described in Section C9.3.3.2C.

Hybrid seismic isolation systems—composed of elastomeric and sliding bearings—should be modeled taking into account the likely significant differences in the relationships between vertical displacement as a function of horizontal displacement. The use of elastomeric and sliding isolators in close proximity to one another under vertically stiff structural framing elements (e.g., reinforced concrete shear walls) may be problematic and could result in significant redistributions of gravity loads.

C9.2.2.3 Modeling of Isolators

A. General

No commentary is provided for this section.

B. Linear Models

For linear procedures (see Section C9.2.3), the seismic isolation system can be represented by an equivalent linearly elastic model. The force in a seismic isolation device is calculated as:

$$F = k_{eff} D \qquad \text{(C9-16)}$$

where all terms are as defined in Section 9.2.2.3B of the *Guidelines*. The effective stiffness of the seismic isolation device may be calculated from test data as follows:

$$k_{eff} = \frac{|F^+| + |F^-|}{|\Delta^+| + |\Delta^-|} \qquad \text{(C9-17)}$$

Figure C9-11 illustrates the physical significance of the effective stiffness.

Analysis by a linear method requires that either each seismic isolator or groups of seismic isolators be represented by linear springs of either stiffness k_{eff} or the combined effective stiffness of each group. The energy dissipation capability of an isolation system is generally represented by effective damping. Effective damping is amplitude-dependent and calculated at design displacement, D, as follows:

$$\beta_{eff} = \frac{1}{2\pi}\left[\frac{\Sigma E_D}{K_{eff} D^2}\right] \qquad \text{(C9-18)}$$

where ΣE_D is the sum of the areas of the hysteresis loops of all isolators, and K_{eff} is the sum of the effective stiffnesses of all seismic isolation devices. Both the area of the hysteresis loops and the effective stiffness are determined at the design displacement, D.

The application of Equations C9-16 through C9-18 to the design of isolation systems is complicated if the effective stiffness and loop area depend on axial load. Multiple analyses are then required to establish bounds on the properties and response of the isolators. For example, sliding isolation systems exhibit such dependencies as described in Section C9.2.2.2B. To account for these effects, the following procedure is proposed.

1. In sliding isolation systems, the relation between horizontal force and vertical load is substantially linear (see Equation C9-12). Accordingly, the net effect of overturning moment on the mechanical

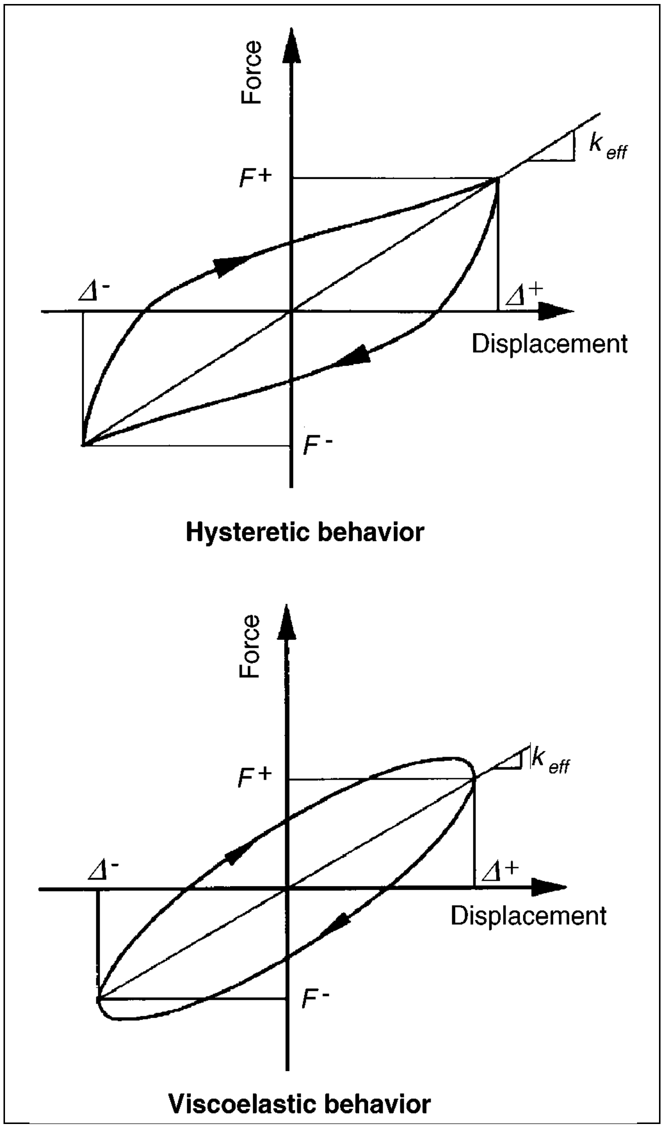


Figure C9-11 Definition of Effective Stiffness of Seismic Isolation Devices

behavior of a group of bearings is small and can be neglected. Al-Hussaini et al. (1994) provided experimental results that demonstrate this behavior up to the point of imminent bearing uplift. Similar results are likely for elastomeric bearings.

2. The effect of vertical ground acceleration is to modify the load on the isolators. If it is assumed that the building is rigid in the vertical direction, and axial forces due to overturning moments are absent, the axial loads can vary between $W(1 - \ddot{U}/g)$ and $W(1 + \ddot{U}/g)$, where $\ddot{U}$ is the peak vertical ground acceleration. However, recognizing that horizontal and vertical ground motion components are likely not correlated unless in the near field, it is appropriate to use a combination rule that uses only a fraction of the peak vertical ground acceleration. Based on the use of 50% of the peak vertical ground acceleration, maximum and minimum axial loads on a given isolator may be defined as:

$$N_c = W(1 \pm 0.20 S_{DS}) \qquad \text{(C9-19)}$$

where the plus sign gives the maximum value and the minus sign gives the minimum value. Equation C9-19 is based on the assumption that the short-period spectral response parameter, S_{DS}, is 2.5 times the peak value of the vertical ground acceleration. For analysis for the Maximum Considered Earthquake, the axial load should be determined from

$$N_c = W(1 \pm 0.20 S_{MS}) \qquad \text{(C9-20)}$$

Equations C9-19 and C9-20 should be used with caution if the building is located in the near field of a major active fault. In this instance, expert advice should be sought regarding correlation of horizontal and vertical ground motion components.

Load N_c represents a constant load on isolators, which can be used for determining the effective stiffness and area of the hysteresis loop. To obtain these properties, the characteristic strength Q (see Figure C9-11) is needed. For sliding isolators, Q can be taken as equal to $f_{max} N_c$, where f_{max} is determined at the bearing pressure corresponding to load N_c. For example, for a sliding bearing with spherical sliding surface of radius R_o (see Figure C9-8), the effective stiffness and area of the loop at the design displacement D are:

$$k_{eff} = \left(\frac{1}{R_o} + \frac{f_{max}}{D}\right) N_c \qquad \text{(C9-21)}$$

$$\text{Loop Area} = 4 f_{max} N_c D \qquad \text{(C9-22)}$$

C. Nonlinear Models

For dynamic nonlinear time-history analysis, the seismic isolation elements should be explicitly modeled. Sections C9.2.2.2 through C9.2.2.4 present relevant information. When uncertainties exist, and when aspects of behavior cannot be modeled, multiple analyses should be performed in order to establish bounds on the dynamic response.

For simplified nonlinear analysis, each seismic isolation element can be modeled by an appropriate rate-independent hysteretic model. Elastomeric bearings may be modeled as bilinear hysteretic elements as described in Section C9.2.2.2. Sliding bearings may also be modeled as bilinear hysteretic elements with characteristic strength (see Figure C9-4) given by

$$Q = f_{max} N_c \qquad \text{(C9-23)}$$

where N_c is determined by either Equation C9-19 or Equation C9-20, and f_{max} is the coefficient of sliding friction at the appropriate sliding velocity. The post-yield stiffness can then be determined as:

$$k_p = \frac{N_c}{R} \qquad \text{(C9-24)}$$

where R is as defined in Section C9.2.2.2B. The yield displacement D_y in a bilinear hysteretic model of a sliding bearing should be very small, perhaps on the order of 2 mm. Alternatively, a bilinear hysteretic model for sliding bearings may be defined to have an elastic stiffness that is at least 100 times larger than the post-yield stiffness k_p.

Isolation devices that exhibit viscoelastic behavior as shown in Figure C9-11 should be modeled as linearly elastic elements with effective stiffness k_{eff} as determined by Equation C9-17.

C9.2.2.4 Isolation System and Superstructure Modeling

A. General

The model (or models) of the isolation system and superstructure serves two primary functions:

1. **Calculation of the BSE-2 displacement of the isolation system.** BSE-2 displacement is used for designing the isolation system, testing isolator prototypes, establishing required clearances, and specifying displacement demand on nonstructural components that cross the isolation interface.

2. **Calculation of the design earthquake response of the structure.** The design earthquake response is used for design of superstructure components and elements, isolation system connections, foundation and other structural components, and elements below the isolation system.

Several approaches can be used for modeling the isolation system and superstructure, ranging from simplified stick models to detailed, three-dimensional finite element models of the entire building. The extent of the modeling will vary depending on the structural configuration, the type of isolation system, and the degree of linearity (or nonlinearity) expected in the superstructure. In general, flexible, irregular, and/or nonlinear superstructures will require more complex modeling.

B. Isolation System Model

The isolation system should be modeled with sufficient detail to accurately determine the maximum displacement of isolators, including the effects of torsion, and to accurately determine forces acting on adjacent structural elements.

The properties of the isolation system (e.g., effective stiffness) may vary due to changes in vertical load, direction of applied load, and the rate of loading. For some systems, properties may change with the number of cycles of load, or otherwise have some significant degree of variability (e.g., as measured during prototype testing). The model of the isolation system will need to explicitly account for the range of isolation system properties, if properties vary significantly (e.g., effective stiffness changes by more than 15% during prototype testing). Typically, two models will need to be used to bound the range of isolation system stiffness. The stiffer isolation system model would be used to calculate superstructure force; the softer isolation system model would be used to calculate isolation system displacement.

Isolation systems can be susceptible to uplift of isolators due to earthquake overturning load. The model of the isolation system should permit uplift of the superstructure to occur, unless the isolators are shown to be capable of resisting uplift force. Uplift is a nonlinear phenomenon and requires either explicit modeling (i.e., vertical gap element) or a linear model that releases vertical load in isolators when the uplift force exceeds the isolator's capacity. It is important that the model permit uplift at isolators, so that the forces in the superstructure redistribute accordingly and the maximum uplift displacement is established for design of the isolation system connections and for testing of isolator prototypes.

Special care must be taken to calculate P-Δ effects because standard analysis procedures typically ignore the effects of the P-Δ moment across isolators. The displacement of the isolation system can create large P-Δ moment on the isolators, the substructure and foundation below, and the superstructure above. Depending on the type of isolator, the P-Δ moment will be at least $(P \text{ times } \Delta)/2$ and may be as great as P times Δ, where P is the axial load in the isolator and Δ is the horizontal isolator displacement. This moment is applied to both the top and the bottom of the isolator interfaces and is in addition to the moment due to shear across the isolator.

C. Superstructure Model

In general, the superstructure should be modeled with as much detail as would be required for a conventional building.

Special care must be taken in modeling the strength and stiffness of the superstructure. The structural system should have the required strength to respond essentially as a linear elastic system, if the superstructure is modeled with elastic elements. The building will not receive the benefit of the isolation system if the superstructure, rather than the isolation system, yields and displaces.

The lateral-force-resisting system of the superstructure may be considered to be essentially linearly elastic, if at each floor the primary elements and components of the lateral-force-resisting system experience limited inelastic demand (i.e., $m \leq 1.5$). Limited inelastic demand would not preclude a few elements or components from reaching the limits established for the material, provided the effective stiffness of the lateral-force-resisting system of the superstructure did not, as a whole, change appreciably.

C9.2.3 General Criteria for Seismic Isolation Design

C9.2.3.1 General

The basis for design should be established using the procedures of Chapter 2 and the building's Rehabilitation Objective(s).

The criteria for design, analysis, and testing of the isolation system are based primarily on requirements for isolation systems of new buildings. This approach acknowledges that the basic requirements for such things as stability of isolators, prototype testing and, quality control, are just as valid for rehabilitation projects as for new construction. A case might be made for less conservative limits on clearances around the isolated building, provided life safety is not compromised. Again, such an argument would not be appropriate for projects with goals dominated by special damage protection or functionality objectives.

Peer review of the isolation system should be performed for all rehabilitation projects, as required by design provisions for new construction. However, the extent of the review should be gauged to the size and importance of the project. Large, important projects require full design and construction review by a panel of seismic isolation, structural, and geotechnical experts, while small projects may be adequately checked by building authorities with only limited oversight by an outside consultant.

Rather than addressing a specific method of base isolation, the *Guidelines* include general design requirements applicable to a wide range of possible seismic isolation systems. In remaining general, the design provisions rely on mandatory testing of isolation system hardware to confirm the engineering properties used in the design and to verify the overall adequacy of the isolation system. Some systems may not be capable of demonstrating acceptability by test, and consequently should not be used. In general, acceptable isolation systems will:

1. Remain stable for the required design displacement

2. Provide increasing resistance with increasing displacement (although some acceptable systems may not fully comply with this provision)

3. Not degrade under repeated cyclic load

4. Have well-defined engineering properties (e.g., established and repeatable force-deflection characteristics)

C9.2.3.2 Ground Shaking Criteria

No commentary is provided for this section.

C9.2.3.3 Selection of Analysis Procedure

The *Guidelines* require either linear or nonlinear procedures for analysis of isolated buildings.

Linear procedures include prescriptive formulas and Response Spectrum Analysis. Linear procedures based on formulas (similar to the seismic-coefficient equation required for design of fixed-base buildings) prescribe peak lateral displacement of the isolation system, and define "minimum" design criteria that may be used for design of a very limited class of isolated structures (without confirmatory dynamic analyses). These simple formulas are useful for preliminary design and provide a means of expeditious review of more complex calculations.

Response Spectrum Analysis is recommended for design of isolated structures that have either (1) a tall or otherwise flexible superstructure, or (2) an irregular superstructure. For most buildings, Response Spectrum Analysis will not predict significantly different displacements of the isolation system than those calculated by prescriptive formulas, provided both calculations are based on the same effective stiffness and damping properties of the isolation system. The real benefit of Response Spectrum Analysis is not in the prediction of isolation system response, but rather in the calculation and distribution of forces in the superstructure. Response Spectrum Analysis permits the use of more detailed models of the superstructure that better estimate forces and deformations of components and elements considering flexibility and irregularity of the structural system.

Nonlinear procedures include the Nonlinear Static Procedure (NSP) and the Nonlinear Dynamic Procedure (NDP). The NSP is a static pushover procedure, and the NDP is based on nonlinear Time-History Analysis. The NSP or the NDP is required for isolated structures that do not have essentially linearly elastic superstructures (during BSE-2 demand). In this case, the superstructure would be modeled with nonlinear elements and components.

Time-History Analysis is required for isolated structures on very soft soil (i.e., Soil Profile Type E when shaking is strong, or Soil Profile Type F) that could shake the building with a large number of cycles of long-period motion, and for buildings with isolation systems that are best characterized by nonlinear models. Such isolation systems include:

1. Systems with more than about 30% effective damping (because high levels of damping can significantly affect higher-mode response of the superstructure)

2. Systems that lack significant restoring force (because these systems may not stay centered during earthquake shaking)

3. Systems that are expected to exceed the sway-space clearance with adjacent structures (because impact with adjacent structures could impose large demands on the superstructure)

4. Systems that are rate- or load-dependent (because their properties will vary during earthquake shaking)

For the types of isolation systems described above, appropriate nonlinear properties must be used to model isolators. Linear properties could be used to model the superstructure, provided the superstructure's response is essentially linearly elastic for BSE-2 demand.

The restrictions placed on the use of linear procedures effectively suggest that nonlinear procedures be used for virtually all isolated buildings. However, lower-bound limits on isolation system design displacement and force are specified by the *Guidelines* as a percentage of the demand prescribed by the linear formulas, even when dynamic analysis is used as the basis for design. These lower-bound limits on key design attributes ensure consistency in the design of isolated structures and serve as a "safety net" against gross underdesign.

C9.2.4 Linear Procedures

C9.2.4.1 General

No commentary is provided for this section.

C9.2.4.2 Deformation Characteristics of the Isolation System

The deformation characteristics of the isolation system should be based on tests of isolator prototypes, as defined in Section 9.2.9. This section not only specifies the type and sequence of prototype testing, but also provides the formulas to be used to develop values of the effective stiffness and effective damping of the isolation system. These formulas acknowledge that effective stiffness and effective damping are, in general, amplitude-dependent and should be evaluated for both design earthquake and BSE-2 levels of response.

The effective stiffness and effective damping of the isolation system are quantities that can (and typically do) vary due to changes in the nature of applied load (e.g., systems that are rate-, amplitude- or duration-dependent). There is also potential for variation between as-designed and as-built values of effective stiffness and damping. Like all products, isolators can only be required to meet design criteria to within certain specified manufacturing tolerances. The intent of the *Guidelines* is to use bounding values of isolation system properties such that the design is conservative for all potential sources of isolation system variability. The *Guidelines* explicitly require design properties to bound measured variations of isolator prototypes, due to the nature of applied load. The *Guidelines* do not explicitly address potential differences between as-designed and as-built properties, placing the responsibility for quality control with the engineer responsible for the structural design (Section 9.2.7.2I).

C9.2.4.3 Minimum Lateral Displacements

A. Design Displacement

Equation 9-2 prescribes design earthquake displacement of the isolation system at the center of mass of the building (pure translation, without contribution from torsion). The equation is based on the effective period (minimum value of effective stiffness) and damping coefficient (minimum value of effective damping) of the isolation system evaluated at the design displacement. The damping coefficient is based on median spectral amplification factors of Table 2 of *Earthquake Spectra and Design* (Newmark and Hall, 1982), as defined in Chapter 2 of the *Guidelines*.

Spectral demand is based on the long-period spectral acceleration coefficient specified in Chapter 2 for the design earthquake (i.e., S_{D1}). Equation 9-2 should be modified for use with site-specific spectral demand by replacing S_{D1}/T_D in this equation with the value of the site-specific design spectrum at the effective period of T_D.

Equation 9-2 effectively calculates push-over displacement of the isolated building, assuming no rotation of the building and a rigid superstructure. The assumption of a rigid superstructure is conservative for estimating isolation system displacement, because any flexibility and displacement of the superstructure would tend to decrease displacement in the isolation system.

B. Effective Period at the Design Displacement

Equation 9-3 prescribes the effective period at the design displacement. The effective period is an estimate of isolated building period based on the secant stiffness of the isolation system at the design displacement. This estimate is conservatively based on the minimum value of effective stiffness, which yields the maximum value of effective period (and hence the largest estimate of building displacement).

C. Maximum Displacement

Equation 9-4 prescribes the BSE-2 displacement of the isolation system. Equation 9-4 is the same as Equation 9-2, except all terms are based on BSE-2 demand and response, rather than design earthquake demand and response.

D. Effective Period at the Maximum Displacement

Equation 9-5 prescribes the effective period of the isolated building at maximum displacement. Equation 9-5 is the same as Equation 9-3, except that effective stiffness is based on BSE-2 displacement, rather than design earthquake displacement.

E. Total Displacement

Isolated systems are required to consider additional displacement due to accidental and actual torsion, similar to the additional loads prescribed for conventional (fixed-base) structures. Equations 9-6 and 9-7 provide a simple estimate of combined translational and torsional displacement based on the gross plan dimensions of the buildings (b and d), the distance from the center of the building to the location of interest, and actual plus accidental eccentricity of the building. Eccentricity is the distance between the center of mass of the superstructure (projected on the plane of the isolation system) and the center of rigidity of the isolation system.

Equations 9-6 and 9-7 are based on the assumption that the stiffness of the isolation system is distributed in a plan proportional to the distribution of supported weight of the superstructure above. This is a reasonable assumption, since most isolator units are designed on the basis of supported weight and tend to be larger (and stiffer) when supporting heavier loads.

Equations 9-6 and 9-7 are evaluated for two bounding cases: (1) a structure that is square in plan, and (2) a structure that is very long in plan in one direction. For these two cases, the additional displacement due to 5% eccentricity is found to be:

1. For structures that are square in plan (i.e., $b = d$):

$$D_{TD}/D_D \text{ or } D_{TM}/D_M = 1.15$$

2. For structures that are long in plan (i.e., $b \gg d$):

$$D_{TD}/D_D \text{ or } D_{TM}/D_M = 1.30$$

The *Guidelines* permit reducing these values if the isolation system is configured to resist torsion (i.e., stiffer isolator units are positioned near the edges and corners of the building), but a minimum value of 10% additional displacement due to torsion is required to provide margin on torsional response.

C9.2.4.4 Minimum Lateral Forces

A. Isolation System and Structural Components and Elements at or below the Isolation System

Equation 9-8 prescribes the lateral force to be used for design of the isolation system, the foundation, and other structural components and elements below the isolation system. Lateral force is conservatively based on the maximum value of effective stiffness of the isolation system evaluated at the design displacement.

B. Structural Components and Elements above the Isolation System

The lateral force to be used for design of the superstructure, V_s, is specified to be the same as that prescribed by Equation 9-8 for design of the isolation system (and structure below). This value of lateral force is based on a conservative estimate of peak force of the design earthquake and corresponds, in concept, to the pseudo lateral load, V, prescribed by Equation 3-6 for linear static analysis of a conventional (fixed-base) building.

C. Limits on V_s

Two lower-bound limits are placed on the design lateral force for the superstructure. The first requirement is intended to keep components and elements of the superstructure elastic for design wind conditions. Design wind loads are not provided with these *Guidelines*, but should be considered as part of the design of an isolated building. Wind will likely not be a factor, unless the design earthquake loads are small.

The second requirement is intended to prevent premature yielding of the superstructure before the isolation system is fully activated. This requirement requires a 1.5 margin between the lateral force to be used for design of the superstructure and the yield level of the isolation system. In the extreme case of a system that has no stiffness after yielding (e.g., flat sliding isolator), the superstructure would be designed for a lateral force that is 50% above the yield level (e.g., 50% above the friction level of the sliding isolator).

D. Vertical Distribution of Force

Equation 9-9 distributes the lateral design force, V_s, over the height of the building on the basis of an inverted triangular force distribution. This distribution has been found to bound response of most isolated buildings conservatively, even when higher modes are excited by hysteretic behavior or large values of effective damping of the isolation system. A less conservative force distribution (e.g., uniform force distribution) would be appropriate for isolation systems that have relatively small values of effective damping, but Time-History Analysis would be required to verify the appropriate distribution of lateral force over the height of the building.

C9.2.4.5 Response Spectrum Analysis

Response Spectrum Analysis should be performed using the procedures described in Section 3.3.2, using effective stiffness and damping properties for the isolation system. The effective stiffness of the isolation system should be the same as that required for use in the linear procedure formulas of Section 9.2.4.3. The effective damping of the fundamental (isolated) mode in each horizontal direction should be the same as that required for use in the linear procedure formulas of Section 9.2.4.3. Damping values for higher modes of response should be consistent with the values specified in Chapter 2 for conventional (fixed-base) buildings.

The Response Spectrum Analysis should produce about the same isolation system displacement and lateral force as those calculated using the linear formulas of Section 9.2.4.3, since the two methods are based on the same effective stiffness and damping properties for the isolation system. Section 9.2.4.5D requires upward scaling of Response Spectrum results, if displacements predicted by Response Spectrum Analysis are less than those of the linear procedure formulas.

C9.2.4.6 Design Forces and Deformations

Components and elements are to be designed 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 $m = 1.5$. Response of structural components and elements is limited to $m = 1.5$ to ensure that the structure remains essentially elastic for the design earthquake. Response of structural components and elements beyond $m = 1.5$ is not recommended without explicit modeling and analysis of building nonlinearity.

C9.2.5 Nonlinear Procedures

C9.2.5.1 Nonlinear Static Procedure

The NSP should follow the push-over methods described in Section 3.3.3, except that the target displacement for the design earthquake is specified by Equation 9-10 and the target displacement for the BSE-2 is specified by Equation 9-11. Target displacements are specified for a control node that is located at the center of mass of the first floor above the isolation system.

Equations 9-10 and 9-11 are based on Equations 9-2 and 9-4, respectively, modified to account for the influence of a flexible superstructure. For isolated buildings with short, stiff superstructures, the isolated period at the design displacement will be several times greater than the effective period of the superstructure (on a fixed base), and the displacement of the isolation system—considering superstructure flexibility—will be about the same as the displacement of the isolation system based on rigid superstructure.

The pattern of applied load should be proportional to the distribution of the product of building mass and the deflected shape of the isolated mode. For isolated buildings with a stiff superstructure (i.e., stiff relative to the isolation system), the deflected shape of the isolated mode is dominated by displacement of the isolation system (e.g., nearly uniform deflected shape). For isolated buildings with a flexible superstructure, the deflected shape is a combination of isolation system and superstructure displacements (e.g., trapezoidal deflected shape).

Isolation systems are typically nonlinear and relatively stiff at low force levels. The deflected shape of such systems is amplitude-dependent and at low levels of ground shaking would be dominated by superstructure displacement. At very low levels of ground shaking, before activation of the isolation system, the deflected shape would appear similar to that of the building on a fixed base (e.g., inverted triangle deflected shape).

C9.2.5.2 Nonlinear Dynamic Procedure

The NDP should follow the time history methods described in Section 3.3.4, except that Section 9.2.5.2B requires upward scaling of time history results, if displacements predicted by Time-History Analysis are less than those of the NSP.

C9.2.5.3 Design Forces and Deformations

No commentary is provided for this section.

C9.2.6 Nonstructural Components

To accommodate the differential movement between the isolated building and the ground, provision for flexible connections should be made. In addition, rigid structures crossing the interface (i.e., stairs, elevator shafts, and walls) should have details that accommodate differential motion at the isolator level without sustaining damage inconsistent with the building's Rehabilitation Objectives.

C9.2.7 Detailed System Requirements

C9.2.7.1 General

No commentary is provided for this section.

C9.2.7.2 Isolation System

No commentary is provided for subsections A through H.

I. Manufacturing Quality Control

A test and inspection program is necessary for both fabrication and installation of the isolation system. Because base isolation is a developing technology, it may be difficult to reference standards for testing and inspection. Reference can be made to standards for some material such as elastomeric bearings (ASTM D4014). Similar standards are required for other isolation systems. Special inspection procedures and load testing to verify manufacturing quality control should be developed for each project. The requirements will vary with the type of isolation system used.

C9.2.8 Design and Construction Review

Design review of both the design and analysis of the isolation system and design review of the isolator testing program are mandated by the *Guidelines* for two key reasons:

1. The consequences of isolator failure could be catastrophic.

2. Isolator design and fabrication is evolving rapidly, and may be based on technologies unfamiliar to many design professionals.

The *Guidelines* require review to be performed by a team of registered design professionals who are independent of the design team and other project contractors. The review team should include individuals with special expertise in one or more aspects of the design, analysis, and implementation of seismic isolation systems.

The review team should be formed prior to the finalization of design criteria (including site-specific ground shaking criteria) and isolation system design options. Further, the review team should have full access to all pertinent information and the cooperation of the design team and authorities having jurisdiction involved with the project.

C9.2.9 Isolation System Testing and Design Properties

C9.2.9.1 General

The isolation system testing procedures of the *Guidelines* represent minimum testing requirements. Other, more extensive testing procedures may be available in the future that would also be suitable for isolation system testing. For example, a standard for testing seismic isolation systems, units, and components is currently being developed by a committee of the American Society of Civil Engineers.

C9.2.9.2 Prototype Tests

All isolator tests should be witnessed and reported by a qualified, independent inspector.

For each cycle of test the force-deflection behavior of the prototype test specimen must be recorded so that data can be used to determine whether the isolation system complies with both the *Guidelines* and specifications prepared by the engineer responsible for design of the structural system. Both the engineer responsible for design and members of the design review team should review all raw data from prototype tests.

Prototype tests are not required if the isolator unit is of similar dimensional characteristics, of the same type and material, and fabricated using the same process as a prototype isolator unit that has been previously tested using the specified sequence of tests. The independent design review team should determine whether the results of previously tested units are suitable, sufficient, and acceptable.

C9.2.9.3 Determination of Force-Deflection Characteristics

No commentary is provided for this section.

C9.2.9.4 System Adequacy

No commentary is provided for this section.

C9.2.9.5 Design Properties of the Isolation System

No commentary is provided for this section.

C9.3 Passive Energy Dissipation Systems

C9.3.1 General Requirements

The *Guidelines* provide systematic procedures for the implementation of energy dissipation devices in seismic rehabilitation. Although these procedures are seminal and mutable, they constitute the first comprehensive suite of such procedures ever published. The procedures set forth in the *Guidelines* will likely change as more information becomes available. The reader is urged to stay abreast of new developments in the field of energy dissipation systems (EDS).

The *Guidelines* provide procedures to calculate member actions and deformations in building frames incorporating energy dissipation devices, and requirements for testing energy dissipation hardware. Component checking for actions and deformations so calculated shall conform with the procedures set forth in Chapter 3 and the strength and deformation limits presented in the materials chapters.

Issues Besides Seismic and Wind Effects

The properties of some energy dissipation devices may change substantially due to wind effects, aging, operating temperature, and high-cycle fatigue. Although these important issues are not addressed in the *Guidelines*, with the exception of typical wind effects, adequate treatment of these issues in the design phase of a project is of paramount importance to ensure reliable performance of the energy dissipation devices. The engineer of record must consider these issues in designing with energy dissipation devices.

New definitions are presented in the *Guidelines* for components associated with energy dissipation devices, namely, support framing for energy dissipation devices, and points of attachment. These components are illustrated in Figure C9-12.

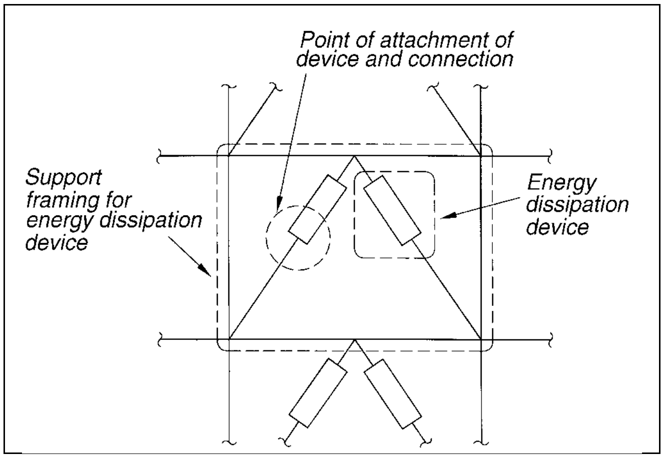


Figure C9-12 Energy Dissipation Nomenclature

The primary reason for introducing energy dissipation devices into a building frame is to reduce the displacements and damage in the frame. Displacement reduction is achieved by adding either stiffness and/or energy dissipation (generally termed *damping*) to the building frame. Metallic-yielding, friction, and viscoelastic energy dissipation devices typically introduce both stiffness and damping; viscous dampers will generally only increase the damping in a building frame. Figure C9-13 simplistically illustrates the impact

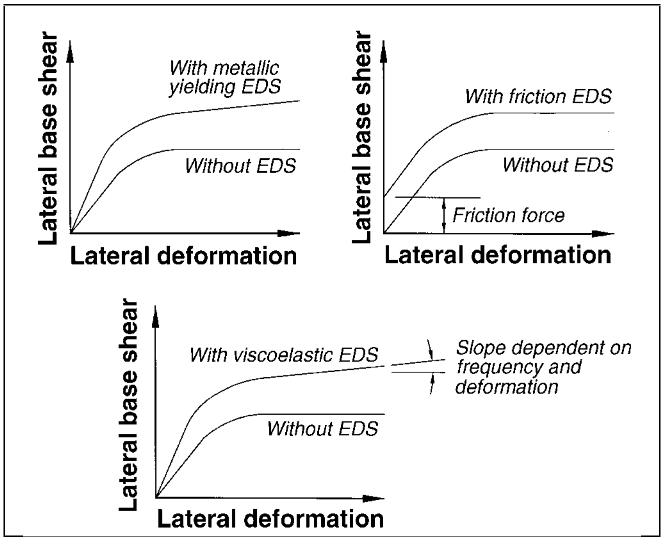


Figure C9-13 Effect of Energy Dissipation on the Force-Displacement Response of a Building

of different types of dampers on the force-displacement response of a building. The addition of viscous dampers will not change the force-displacement relation; that is, the "with viscous EDS" curve will be essentially identical to the "without EDS" curve in Figure C9-13.

As noted above, the force-displacement relation for selected types of energy dissipation devices may be dependent on environmental conditions (e.g., wind, aging, and operating temperature), and excitation frequency, sustained deformations, and bilateral deformations. Such dependence should be investigated by analysis of the mathematical model with limiting values assigned to the properties of the energy dissipation devices.

The Analysis Procedures set forth in the *Guidelines* are approximate only. Roof displacements calculated using the linear and nonlinear procedures are likely to be more accurate than the corresponding estimates of inter-story drift and relative velocity between adjacent stories. Accordingly, the *Guidelines* require that energy dissipation devices be capable of sustaining larger displacements (and velocities for velocity-dependent devices) than the maxima calculated by analysis in the BSE-2. Recognizing that the response of a building frame incorporating four or more devices in each principal direction in each story will be more reliable than a frame with fewer devices in each principal direction, the increase in displacement (and velocity) capacity is dependent on the level of redundancy in the supplemental damping system. The increased force shall be used to design the framing that supports the energy dissipation devices—reflecting the objective of keeping the device support framing elastic in the BSE-2. The increases in force and displacement capacity listed in the *Guidelines* (= 130% for four or more devices and 200% for fewer than four devices) are based on the judgment of the authors at the time of this writing.

The *Guidelines* require that the stiffness characteristics of the energy dissipation devices and the device support framing be included in the mathematical model of the building. If the stiffness of the support framing is ignored, the lateral stiffness of the building may be substantially underestimated (and the target displacements significantly overestimated). Conversely, if flexible support framing is assumed to be rigid, the effectiveness of the dampers may be overestimated, leading to nonconservative results. The reader is referred to Constantinou et al. (1996) for additional information.

C9.3.2 Implementation of Energy Dissipation Devices

Restrictions on the use of linear procedures are established in Chapter 2. These restrictions also apply to the implementation of energy dissipation devices using linear procedures.

At the time of this writing, the use of linear procedures for implementing energy dissipation devices is limited to buildings in which all components and elements, exclusive of the energy dissipation devices, remain in the linearly elastic range for the BSE-2. Calculation of component actions should reflect the benefits of the added damping. There are no limits on the use of nonlinear procedures except for the restrictions set forth in Chapter 2.

It must be emphasized that linear procedures are only appropriate for linearly elastic buildings incorporating viscoelastic or viscous energy dissipation devices. However, if the level of equivalent viscous damping is small (less than 30% of critical), hysteretic energy dissipation devices can be treated as viscous devices. Procedures for implementing both hysteretic (displacement-dependent) devices and viscous and viscoelastic (velocity-dependent) devices are presented in Section 9.3.4.1.

Given the similarity between metallic-yielding devices and shear links in eccentrically braced steel frames, consideration was given to developing linear procedures for implementing metallic-yielding devices in framing systems permitted to undergo inelastic response. However, the authors were unable to develop robust rules linking the minimum yielding strength of the energy dissipation devices to the yielding strength of the existing framing—a key step in limiting the degree of inelastic action in the existing framing. Accordingly, no such linear procedures were included in the *Guidelines*.

C9.3.3 Modeling of Energy Dissipation Devices

The *Guidelines* identify three types of energy dissipation devices: displacement-dependent, velocity-dependent, and "other." Metallic-yielding and friction dampers are classed as displacement-dependent devices. Figure C9-14 shows sample force-displacement relations for displacement-dependent devices. Shape-memory alloy dampers can be configured to produce hysteretic response similar to that shown in Figure C9-14.

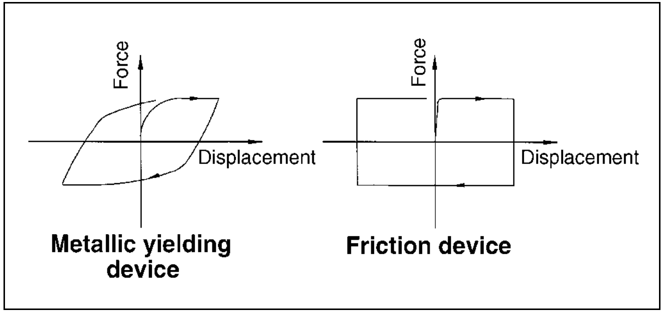


Figure C9-14 Idealized Force-Displacement Loops of Displacement-Dependent Energy Dissipation Devices

Examples of velocity-dependent energy dissipation devices include viscoelastic solid dampers, dampers operating by deformation of viscoelastic fluids (e.g., viscous shear walls), and dampers operating by forcing a fluid through an orifice (e.g., viscous fluid dampers). Figure C9-15 illustrates the typical behavior of these devices.

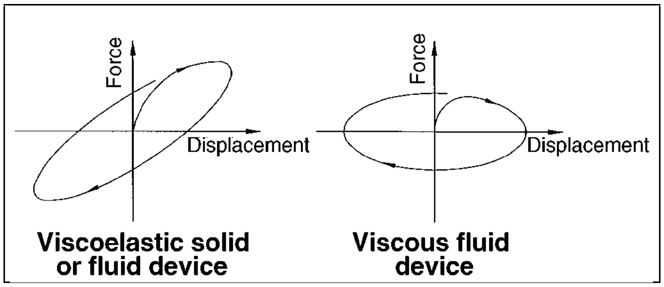


Figure C9-15 Idealized Force-Displacement Loops of Velocity-Dependent Energy Dissipation Devices

Other devices have characteristics that cannot be classified by either of the basic types depicted in Figures C9-14 and C9-15. Examples are devices made of shape-memory alloys, friction-spring assemblies with recentering capability, and fluid restoring force-damping devices. Figure C9-16 presents force-displacement relations for these devices, which dissipate energy while providing recentering capability, and resist motion with a nearly constant force. Shape-memory alloy devices may be designed to exhibit behavior of the type shown in Figure C9-16. The reader is referred to ATC (1993), EERI (1993), and Soong and Constantinou (1994) for more information.

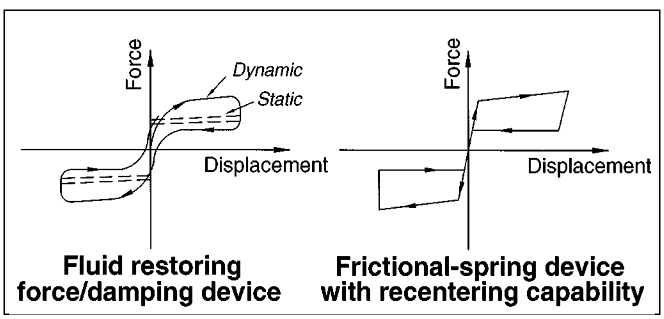


Figure C9-16 Idealized Force-Displacement Loops of Energy Dissipation Devices with Recentering Capability

C9.3.3.1 Displacement-Dependent Devices

Displacement-dependent devices exhibit bilinear or trilinear hysteretic, elasto-plastic or rigid-plastic (frictional) behavior. Details on the behavior and modeling of such devices may be found in Whittaker et al. (1989), Aiken and Kelly (1990), ATC (1993), Soong and Constantinou (1994), Grigorian and Popov (1994), Yang and Popov (1995), and Li and Reinhorn (1995).

C9.3.3.2 Velocity-Dependent Devices

A. Solid Viscoelastic Devices

Solid viscoelastic devices typically consist of constrained layers of viscoelastic polymers. Such devices exhibit viscoelastic solid behavior with mechanical properties dependent on frequency, temperature, and amplitude of motion. A sample force-displacement relation for a viscoelastic solid device under sinusoidal motion of circular frequency, ω, is shown in Figure C9-17. The force may be expressed as:

$$F = k_{eff} D + C\dot{D} \qquad \text{(C9-25)}$$

where all terms are as defined in Section 9.3.3.2 of the *Guidelines*. The effective stiffness of the energy dissipation device is calculated as:

$$k_{eff} = \frac{|F^+| + |F^-|}{|D^+| + |D^-|} \qquad \text{(C9-26)}$$

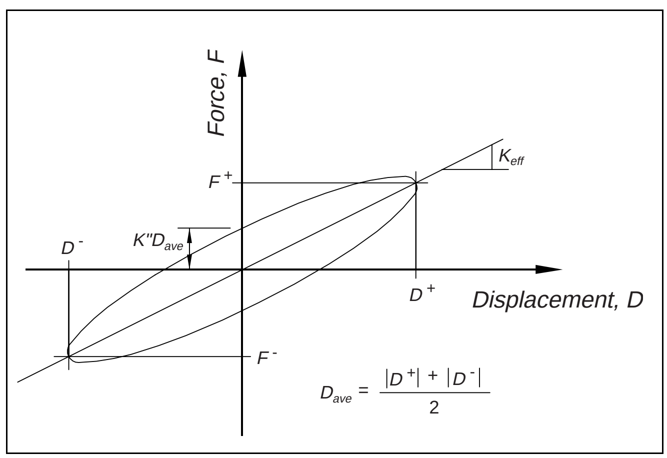


Figure C9-17 Idealized Force-Displacement Relation for a Viscoelastic Solid Device

and the damping coefficient C of the device is calculated as:

$$C = \frac{W_D}{\pi \omega D_{ave}^2} \qquad \text{(C9-27)}$$

where D_{ave} is the average of the absolute values of D^+ and D^-; and W_D is the area enclosed by one complete displacement cycle (D^+ to D^-) of the device.

The effective stiffness is also termed the storage shear stiffness, K' in the literature. The damping coefficient can be described in terms of the loss stiffness, K'':

$$C = \frac{K''}{\omega} \qquad \text{(C9-28)}$$

The effective stiffness and damping coefficient are generally dependent on the frequency, temperature, and amplitude of motion. Figure C9-18 shows normalized values of these parameters from the tests of Chang et al. (1991) of one viscoelastic polymer. Shear strains γ are identified. Note that the frequency and temperature dependence of viscoelastic polymers tend to vary as a function of the composition of the polymer (Bergman and Hanson, 1993). The results presented in Figure C9-18 are not indicative of all viscoelastic solids. The normalized parameters in this figure are the storage shear modulus (G') and loss shear modulus (G'').

Viscoelastic solid behavior can be modeled over a wide range of frequencies using advanced models of viscoelasticity (Kasai et al., 1993). Simpler models are capable of capturing behavior over a limited frequency range—these models will suffice for most rehabilitation projects. For example, the standard linear solid model shown in Figure C9-19 was used to model the behavior of the device of Figure C9-18 at temperature of 21°C. The results presented in Figure C9-20 were obtained using values of $G_1 = 5.18\ \text{MPa}$, $G_2 = 0.48\ \text{MPa}$, and $\eta_2 = 0.31$ MPa-sec/rad where

$$G_1 = \frac{K_1 t}{A_b} \qquad G_2 = \frac{K_2 t}{A_b} \qquad \eta_2 = \frac{C_2 t}{A_b} \qquad \text{(C9-29)}$$

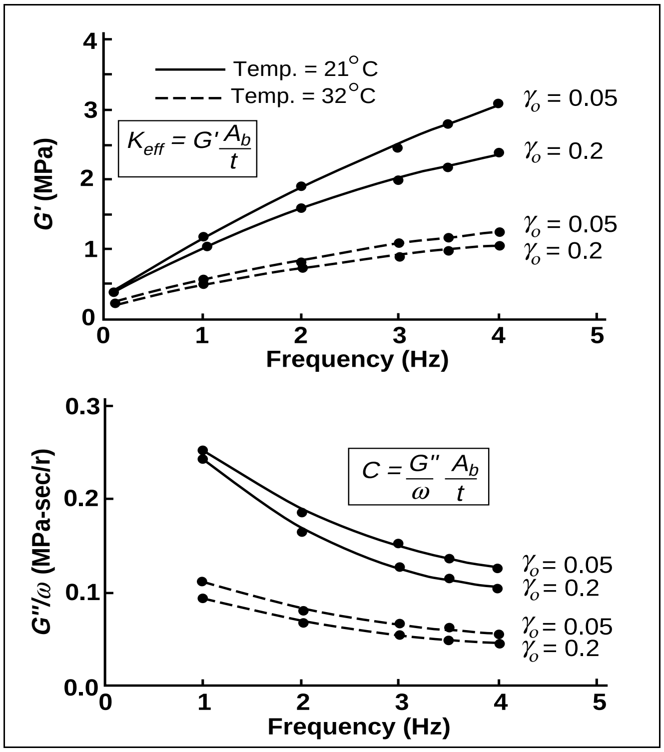


Figure C9-18 Normalized Effective Stiffness (G') and Damping Coefficient (G''/ω) of Viscoelastic Solid Device

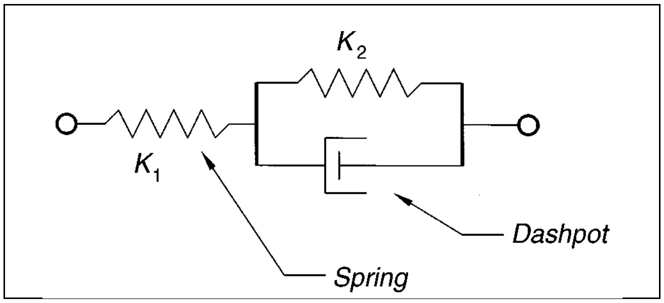


Figure C9-19 Model for Viscoelastic Energy Dissipation Device Behavior

In the above formulae, K_1, K_2, and C_2 are the spring and dashpot constants for the standard linear solid model, A_b is the bonded area of the device, and t is the thickness of viscoelastic material in the device.

B. Fluid Viscoelastic Devices

Fluid viscoelastic devices, which operate by shearing viscoelastic fluids (ATC, 1993), have behaviors that resemble those of solid viscoelastic devices

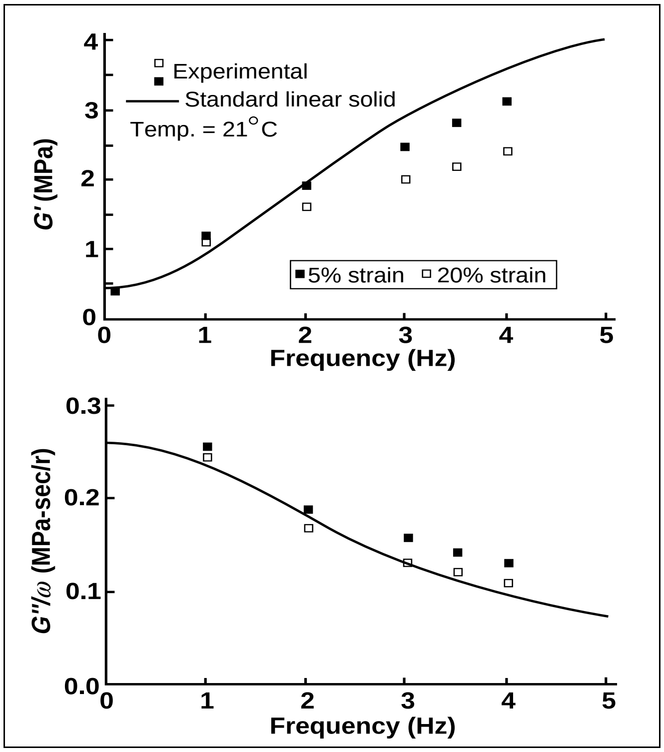


Figure C9-20 Properties of Viscoelastic Solid Device Predicted by Standard Linear Solid Model

(Figure C9-14), except that fluid viscoelastic devices have zero effective stiffness under static loading. Fluid and solid viscoelastic devices are distinguished by the ratio of loss stiffness to effective stiffness as the loading frequency approaches zero: the ratio approaches infinity for fluid viscoelastic devices, and zero for solid viscoelastic devices.

Fluid viscoelastic behavior can be modeled with advanced models of viscoelasticity (Makris et al., 1993). However, fluid viscoelastic devices can be modeled using the Maxwell model of Figure C9-21 in most instances.

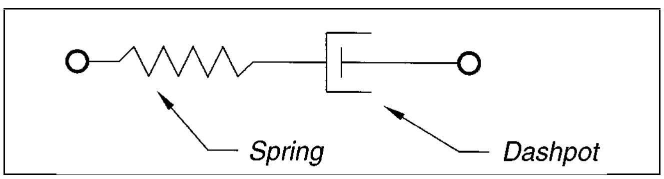


Figure C9-21 Maxwell Model for Fluid Viscoelastic Energy Dissipation Devices

C. Fluid Viscous Devices

Pure viscous behavior can be produced by forcing fluid through an orifice (Constantinou and Symans, 1993; Soong and Constantinou, 1994). Fluid viscous devices may exhibit some stiffness at high frequencies of cyclic loading. Linear fluid viscous dampers exhibiting stiffness in the frequency range $0.5f_1$ to $2.0f_1$ should be modeled as fluid viscoelastic devices, where f_1 is the fundamental frequency of the rehabilitated building.

The frequency range of 0.5 f_1 to 2.0 f_1 is used throughout Section 9.3. The lower limit of 0.5 f_1 corresponds to a fourfold reduction in lateral stiffness; such a reduction is likely an upper bound due to the limited deformation capacity assigned to existing construction. The upper limit of 2.0 f_1 recognizes that building components and elements that are not included in the mathematical model may contribute substantial stiffness, producing a larger value of f_1.

In the absence of stiffness in the frequency range $0.5f_1$ to $2.0f_1$, the force F in a fluid viscous device may be calculated as:

$$F = C_0 |\dot{D}|^{\alpha} \operatorname{sgn}(\dot{D}) \qquad \text{(C9-30)}$$

where the terms are as defined in Section 9.3.3.2 of the *Guidelines*. The simplest form of the fluid viscous damper is the linear fluid damper, for which the exponent α is equal to 1.0. Typical values for α range between 0.5 and 2.0.

C9.3.3.3 Other Types of Devices

Other energy dissipating devices, such as those having hysteresis of the type shown in Figure C9-16, require modeling techniques different from those described above. Tsopelas and Constantinou (1994), Nims et al. (1993), and Pekcan et al. (1995) describe analytical models for some of these devices.

C9.3.4 Linear Procedures

General linear procedures for analysis of rehabilitated buildings incorporating energy dissipation devices have not been developed to the level necessary for inclusion in the *Guidelines*, except for rehabilitated framing systems that are specifically designed to remain linearly elastic for the design earthquake.

The stiffness of the energy dissipation devices and their support framing should be included in the mathematical model to adequately capture the dynamic characteristics of the rehabilitated building. Ignoring the influence of added stiffness of the energy dissipation assembly to the rehabilitated building could lead to: spectral displacement demands being overestimated, spectral force demands being underestimated, and modal damping coefficients being calculated incorrectly. Secant stiffness should be used to linearize the energy dissipation devices; this assumption is conservative, because displacements will be overestimated and the benefits of the damping added by the devices will be underestimated.

The mathematical model of the rehabilitated building should account for both the plan and vertical spatial distribution of the energy dissipation devices to enable explicit evaluation of load paths and design actions in components surrounding the energy dissipation assembly.

Velocity-dependent energy dissipation devices may be dependent on loading frequency, temperature, deformation (or strain), velocity, sustained loads, and bilateral loads. Such dependence should be accounted for in the analysis phase by multiple analyses of the rehabilitated building using bounding values of the dependent properties.

C9.3.4.1 Linear Static Procedure

A. Displacement-Dependent Devices

Two additional restrictions on the use of Linear Static Procedures for implementing displacement-dependent energy dissipation devices are set forth in Section 9.3.4.1. The first restriction:

> "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 said ratio. The maximum story resistance shall include the contributions from all components, elements, and energy dissipation devices."

is intended to ensure somewhat uniform yielding of the stories in the building frame and to avoid the concentration of damage in any one story. Plastic analysis by story of the building frame (including the energy dissipation devices) is the preferred method of calculating the maximum resistance of each story.

The second restriction:

> "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."

is intended to limit the influence of the energy dissipation devices on the response of the rehabilitated building. In short, the second restriction limits the resistance of the energy dissipation devices in any story to one-third of the total resistance of the building frame (including the energy dissipation devices) in that story.

Subject to the limit of 30% total equivalent viscous damping in the rehabilitated building, the added damping afforded by the displacement-dependent devices is used to reduce the pseudo lateral load of Equation 3-6 using the damping modification factor of Table 2-15. The calculation of the damping effect should be estimated as follows:

1. Estimate the modified pseudo lateral load by reducing the pseudo lateral load V of Equation 3-6 by the damping modification factor, B, either B_s or B_1, of Table 2-15 corresponding to the assumed effective damping in the rehabilitated building.

2. Calculate the horizontal forces, F_x, from Equations 3-7 and 3-8 using the modified V in lieu of the V.

3. Calculate the horizontal displacements δ_i at each floor level i by linear analysis of the mathematical model using the horizontal forces F_x.

4. Using the displacements δ_i, estimate the effective damping, β_{eff}, as follows:

$$\beta_{eff} = \beta + \frac{\sum_j W_j}{4\pi W_k} \qquad \text{(C9-31)}$$

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:

$$W_k = \frac{1}{2}\sum_i F_i \delta_i \qquad \text{(C9-32)}$$

where all terms are defined above and the summation extends over all floor levels i.

5. Iterate on steps 1 through 4 until the estimate of the effective damping used to calculate the modified equivalent base (used in step 1) is equal to the effective damping calculated in the subsequent step 4.

B. Velocity-Dependent Devices

One additional restriction on the use of Linear Static Procedures for implementing velocity-dependent energy dissipation devices is set forth in Section 9.3.4.1. The restriction:

> "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."

is intended to limit the influence of the energy dissipation devices on the response of the rehabilitated building. In short, the restriction limits the resistance of the energy dissipation devices in any story to one-third of the total resistance of the building frame (including the energy dissipation devices) in that story.

Subject to the limit of 30% total equivalent viscous damping in the rehabilitated building, the added damping afforded by the velocity-dependent devices is used to reduce the pseudo lateral load of Equation 3-6 using the damping modification factor of Table 2-15. The calculation of the damping effect should be estimated as follows:

1. Estimate the modified pseudo lateral load V by reducing V of Equation 3-6 by the damping modification factor, B, either B_s or B_1, of Table 2-15 corresponding to the assumed effective damping in the rehabilitated building.

2. Calculate the horizontal forces, F_x, from Equations 3-7 and 3-8 using the modified V in lieu of V.

3. Calculate the horizontal displacements δ_i at each floor level i by linear analysis of the mathematical model using the horizontal forces F_x.

4. Using the displacements δ_i, estimate the effective damping, β_{eff}, as follows:

$$\beta_{eff} = \beta + \frac{\sum_j W_j}{4\pi W_k} \qquad \text{(C9-33)}$$

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 C9-34:

$$W_k = \frac{1}{2}\sum_i F_i \delta_i \qquad \text{(C9-34)}$$

where all terms are as defined above. The work done by device j in one complete cycle of loading may be calculated as:

$$W_j = \frac{2\pi^2}{T} C_j \delta_{rj}^2 \qquad \text{(C9-35)}$$

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.

5. Iterate on steps 1 through 4 until the estimate of the effective damping used to calculate the modified

equivalent base (used in step 1) is equal to the effective damping calculated in the subsequent step 4.

The calculation of actions in components of a rehabilitated building with velocity-dependent energy dissipation devices is complicated because the viscous components of force are not directly accounted for. Section 9.3.4.1 describes three possible stages of deformation that may result in the maximum member actions: (1) the stage of maximum drift at which the viscous forces are zero, (2) the stage of maximum velocity at which drifts are zero, and (3) the stage of maximum acceleration.

Viscous forces are maximized at the time of maximum velocity. The horizontal components of these viscous forces are balanced by inertia forces such that the resultant lateral displacements are zero. The viscous forces will introduce axial forces into columns supporting the viscous dampers. The magnitude of these axial forces will be dependent on (a) the amount of damping added by the viscous devices, and (b) the number of dampers used to achieve the target level of additional damping.

The time of maximum acceleration is determined assuming that the building undergoes harmonic motion at frequency f_1 and amplitude corresponding to the maximum drift. Under this condition, the maximum acceleration is equal to the acceleration at maximum drift times ($CF_1 + 2\ \beta_{eff} CF_2$). Constantinou et al. (1996) demonstrate that this assumption produces results of acceptable accuracy. Note that the use of $CF_1 = CF_2 = 1$ will result in conservative estimates of component action.

C9.3.4.2 Linear Dynamic Procedure

The primary effect of the added damping and stiffness provided by the energy dissipation devices is a reduction in displacements due to (1) a reduction in the fundamental period, and (2) smaller spectral demands due to additional damping.

The lower-bound limit on the actions and displacements calculated using the linear Response Spectrum Method (= 80% of those actions and deformations estimated using the Linear Static Procedure) is included to guard against inappropriate or incorrect use of dynamic analysis.

A. Displacement-Dependent Devices

Equation 9-26 may be modified to calculate modal damping ratios using modal estimates of the work done by the devices and estimates of the modal strain energy. Recognizing that the displacement of a rehabilitated building will be dominated by first mode response, one strategy worthy of consideration is that which modifies the first mode damping ratio to reflect the additional energy dissipation provided by the dampers, and ignores the benefits of the energy dissipators in reducing response in the higher modes.

B. Velocity-Dependent Devices

Equations 9-33 through 9-35 may be used to calculate modal damping ratios that will account for the additional damping afforded by the energy dissipation devices. The spectral demands should be estimated using the revised estimates of modal damping. Given that the displacement of a rehabilitated building will be dominated by first mode response, one strategy worthy of consideration is that which modifies the first mode damping ratio to reflect the additional energy dissipation provided by the dampers, and ignores the benefits of the energy dissipators in reducing response in the higher modes.

C9.3.5 Nonlinear Procedures

C9.3.5.1 Nonlinear Static Procedure

Section 3.3.3 of the *Guidelines* presents one procedure for nonlinear static analysis. The commentary to this section denotes this procedure as Method 1. An alternative procedure, termed Method 2, is described in Section C3.3.3.3.

Procedures for implementing energy dissipation devices using both Methods 1 and 2 are presented below. The key difference between the methods is the procedure used to calculate the target displacement. Method 1 calculates the target displacement using a series of coefficients and an estimate of the elastic first mode displacement of the building. Method 2 is an iterative procedure that calculates the target displacement as the intersection of a “spectral capacity curve” (conceptually similar to the pushover curve) and a “design demand curve.” The design demand curve is derived from the elastic response spectrum using a level of viscous damping consistent with the energy dissipated by the building in one cycle of loading to the assumed target displacement. There is no preferred method for the implementation of energy dissipation devices. There is no difference between the methods once the target displacement is calculated.

Method 1

A. Displacement-Dependent Devices

The benefit of adding displacement-dependent energy dissipation devices is evidenced by the increase in building stiffness afforded by such devices, and the reduction in target displacement associated with the reduction in T_e. No direct account is taken of the added damping provided by the energy dissipation devices.

The calculation of the target displacement is based on a statistical relationship between the displacement of an elastic single-degree-of-freedom (SDOF) oscillator and the displacement of the corresponding inelastic oscillator—recognizing that the hysteretic energy dissipated by the inelastic oscillator reduces the displacement to that of the elastic oscillator. As such, the hysteretic energy dissipated by a displacement-dependent damper is conceptually identical to that dissipated by a shear link in an eccentrically braced frame. For the latter system, no direct account is taken of the energy dissipated by the shear link for the calculation of the target displacement. Rather, the increase in stiffness and reduction in period due to the addition of the braced framing results in substantially smaller displacement demands. The same rationale applies to displacement-dependent energy dissipation devices.

B. Velocity-Dependent Devices

The target displacement should be reduced to account for the damping added by the velocity-dependent energy dissipation devices. The calculation of the damping effect may be estimated as follows:

1. Estimate the effective damping in the rehabilitated building, including the damping provided by the energy dissipation devices.

2. Calculate the modified target displacement using Equation 3-11 and the damping modification factor from Table 2-15 corresponding to the effective damping calculated in step 1.

3. Impose lateral forces on the mathematical model of the rehabilitated building until the target displacement is reached. Tabulate the horizontal loads (= F_i at floor level i) and horizontal displacements (= δ_i at floor level i) at each floor level at the target displacement. Tabulate the relative axial displacements between the ends of each energy dissipation device (= δ_{rj} for device j)

4. Using the displacements δ_i, estimate the effective damping (β_{eff}) as follows:

$$\beta_{eff} = \beta + \frac{\sum_j W_j \cos^2\theta_j}{4\pi W_k} \quad \text{(C9-36)}$$

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, θ_j is the angle of inclination of device j to the horizontal, and W_k is the maximum strain energy in the frame, determined using Equation C9-37:

$$W_k = \frac{1}{2}\sum_i F_i \delta_i \quad \text{(C9-37)}$$

where all terms are as defined above. The work done by device j in one complete cycle of loading may be calculated as:

$$W_j = \frac{2\pi^2}{T_s} C_j \delta_{rj}^2 \quad \text{(C9-38)}$$

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. Procedures to calculate the work done by a nonlinear viscous damper in one cycle of loading are given in the following discussion on Method 2. (Note that the Method 2 discussion uses global frame displacements, Δ, and not the local component displacements, δ, used above.)

5. Iterate on steps 1 through 4 until the estimate of the effective damping (β_{eff}) used to calculate the modified target displacement (used in step 2) is equal to the effective damping calculated in the subsequent step 4.

The maximum actions in the building frame should be calculated at three stages: maximum drift, maximum velocity, and maximum acceleration. Calculation of component actions and deformations at the time of maximum drift is routine. Similar calculations of component actions and deformations at the times of maximum velocity and maximum acceleration are more complicated and will generally require additional modal analysis. One such procedure is illustrated by example in Section C9.3.9.5, Figure C9-31; the steps in the procedure are enumerated below. This procedure can be used with both Methods 1 and 2.

1. Estimate the secant stiffness of each component and element in the building frame at the target displacement. Replace the elastic stiffness of each component and element with the calculated secant stiffness. Perform eigenvalue analysis of the building frame and identify modal frequencies and shapes. (The first mode period should be similar to the secant period.) Using the design response spectrum, perform Response Spectrum Analysis using these frequencies and shapes, and calculate the maximum roof displacement using a modal combination rule (e.g., SRSS). Scale the modal displacements by the ratio of the target displacement to the maximum roof displacement to update the modal displacements. These modal data would correspond to the floor displacements listed in lines 4 through 6 of Table C9-10.

2. Calculate the modal actions in each component and element at the time of maximum drift. Combine these actions using a modal combination rule. This modal information would correspond to the first-story column actions listed in lines 16 and 17 of Table C9-10.

3. Calculate the modal viscous forces in each velocity-dependent energy dissipation device using modal relative displacements and modal frequencies.

4. For each mode of response, apply the calculated modal viscous forces to the mathematical model of the building at the points of attachment of the devices and in directions consistent with the corresponding mode shape of the building.

5. For each mode of response, apply the horizontal inertia forces at each floor level of the building to the mathematical model concurrently with the modal viscous forces so that the horizontal displacement at each floor level is zero.

6. Calculate the modal component actions resulting from the application of the modal viscous and inertia forces. Combine these actions using a modal combination rule. This modal information would correspond to the first-story column actions listed in line 18 of Table C9-10.

7. Calculate modal component actions for checking at the time of maximum acceleration as the linear combination of component actions due to displacement (step 2) multiplied by factor CF_1 and component actions due to viscous effects (step 6) multiplied by factor CF_2. For each mode of response, factors CF_1 and CF_2 should be calculated using (a) the effective modal damping ratio, and (b) Equations 9-31 and 9-32. The resulting modal component actions should be combined by an appropriate rule to calculate component actions for design. Component actions for design shall equal or exceed the component actions due to displacement. This modal information would correspond to the first-story column actions listed in lines 19 and 20 of Table C9-10.

8. Calculate the component actions for design as the maximum value of the component actions estimated at the times of maximum drift, maximum velocity, and maximum acceleration.

The acceptance criteria of Section 3.4.3 apply to buildings incorporating energy dissipation devices. Checking for displacement-controlled actions shall use deformations corresponding to the target displacement and maximum component forces. Checking for force-controlled actions shall use maximum component actions determined in step 8 above. Evaluation of the energy dissipation devices should be based on experimental data.

The commentary to Section 3.3.3 provides information on two Nonlinear Static Procedures. The procedures described above are intended for use with the nonlinear procedure presented in Section 3.3.3 and are described as Method 1 in the commentary to Section 3.3.3. The second procedure, termed Method 2 in the commentary to Section 3.3.3, may also be used to implement energy dissipation devices. The reader is referred to the following commentary for information on how to use Method 2 to implement passive energy dissipation devices.

Method 2

The target displacement of the rehabilitated building is obtained in Method 2 by the spectral capacity curve (a property of the rehabilitated building) on the design demand curve. The spectral capacity curve is developed using the base shear-roof displacement relation of the rehabilitated building. The design demand curve is established from the 5%-damped pseudo-acceleration response spectrum after adjustment for the effective damping of the rehabilitated building due to inelastic action in the seismic framing system exclusive of the energy dissipation devices, and the added damping provided by the energy dissipation devices.

Design Demand Curve. The 5%-damped response spectrum (spectra) should be developed using the procedures set forth in Chapter 2.

To apply Method 2 to rehabilitated buildings with energy dissipation devices, the 5%-damped spectrum is modified to account for the damping in the rehabilitated building. The spectrum is modified by multiplying the 5%-damped spectral acceleration ordinates by the damping modification factors B_S or B_1, which vary with period range and damping level from Table 2-15. Figure C9-22 illustrates the construction of such a response spectrum from the corresponding 5%-damped spectrum. The modified design demand curve is prepared by constructing the spectral acceleration versus spectral displacement relation for the rehabilitated building at the damping level corresponding to the effective damping of the rehabilitated building.

Given that this simplified method of nonlinear analysis is based in part on modal analysis, a brief review of modal analysis theory is provided below. The reader is referred to Chopra (1995) for additional information.

Modal Analysis Theory. Consider a building represented by reactive weights W_i lumped at N degrees-of-freedom (DOF). The key dynamic

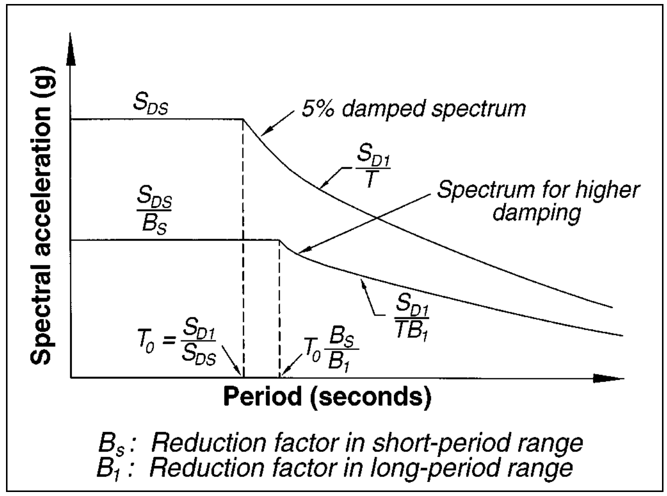


Figure C9-22 ***Construction of Response Spectrum for Damping Higher than 5%***

characteristics of the building are the natural periods and the mode shapes. For this discussion, the amplitude of the m-th mode shape at DOF i is designated as ϕ_{im}. The building can be represented by a single DOF system with weight equal to:

$$W_{sm} = \frac{\left(\sum_{i=1}^{N} W_i \phi_{im}\right)^2}{\sum_{i=1}^{N} W_i \phi_{im}^2} \qquad \text{(C9-39)}$$

Note that the m-th modal weight is less than the total weight of the building and the sum of all the modal weights equals the total weight of the building.

If the spectral acceleration and displacement responses of this single DOF system are denoted as S_{am} and S_{dm}, respectively, the contribution of the m-th mode to the peak response of the building is:

Base shear:

$$V_m = \frac{W_{sm} S_{am}}{g} \qquad \text{(C9-40)}$$

Displacement at DOF i:

$$\delta_{im} = \phi_{im} \Gamma_m S_{dm} \qquad \text{(C9-41)}$$

where Γ_m is the mth modal participation factor:

$$\Gamma_m = \frac{\sum_{i=1}^{N} W_i \phi_{im} S_i}{\sum_{i=1}^{N} W_i \phi_{im}^2} \qquad \text{(C9-42)}$$

The term S_i in Equation C9-42 is the horizontal displacement at DOF i corresponding to a unit horizontal ground displacement. For a two-dimensional mathematical model, S_i is equal to 1.0.

Spectral Capacity Curve. The force-displacement relation from the NSP is manipulated to produce the push-over curve for the building. The push-over curve is typically presented in terms of base shear (ordinate) and roof displacement (abscissa).

To determine whether the design of a rehabilitated building is acceptable, the spectral capacity curve is overlain on the design demand spectrum. The spectral capacity curve is typically presented as spectral acceleration (S_a) versus spectral displacement (S_d). The spectral capacity curve can be derived from the push-over curve of the rehabilitated building by the following procedure.

1. At selected increments of displacement in the push-over analysis, the fundamental mode shape of the rehabilitated building is determined. This mode shape can be evaluated by either: (a) performing an eigenvalue analysis of the building using the secant stiffness of the framing members at the selected level of displacement, or (b) selecting a mode shape with ordinates equal to the displacements at the selected level of displacement; that is, at DOF i, the modal ordinate ϕ_i is equal to δ_i. Method (b) is often used for the Ritz analysis of complex dynamic systems (Chopra, 1995).

2. The spectral acceleration is computed as:

$$S_a = \frac{V}{W_{sm}} g \qquad \text{(C9-43)}$$

   where V is the base shear computed in the NSP; and W_{sm} is calculated per Equation C9-39 using the assumed mode shaped ordinates.

3. The spectral displacement is computed as:

$$S_d = \frac{\delta_t}{\phi_{rm} \Gamma_m} \qquad \text{(C9-44)}$$

   where δ_r is the roof displacement computed in the pushover analysis, ϕ_{rm} is the amplitude of the mode shape at the roof, and Γ_m is the modal participation factor calculated for the assumed mode shape per Equation C9-42.

Effective Damping of Rehabilitated Building. The effective damping of the rehabilitated building must be calculated in order to construct the design demand curve. In general, the effective damping will be dependent on the level of deformation in the framing system.

The effective damping is defined as:

$$\beta_{eff} = \frac{W_D}{4\pi W_k} \qquad \text{(C9-45)}$$

where W_D is the energy dissipated by the rehabilitated building (including the energy dissipation devices) in one complete cycle of motion. The term W_k is the strain energy stored in the rehabilitated building at displacements equal to those used to estimate W_D.

In the push-over analysis, lateral loads F_i (typically a function of a selected displacement quantity) are applied at each reactive weight (W_i), resulting in corresponding displacements δ_i. The strain energy can be estimated as:

$$W_k = \frac{1}{2}\sum_{i=1}^{N} F_i\delta_i \qquad \text{(C9-46)}$$

The dissipated energy should be calculated for a complete cycle of motion at displacements equal to those used to calculate the strain energy, as follows:

$$W_D = W_{DS} + W_{DE} \qquad \text{(C9-47)}$$

where W_{DS} is the energy dissipated by the framing system exclusive of the energy dissipation system (typically assumed to be hysteretic), and W_{DE} is the energy dissipated by the energy dissipation devices, which may be either displacement-dependent or velocity-dependent. For velocity-dependent energy dissipation devices, the dissipated energy should be calculated for one cycle of motion of roof displacement amplitude δ_r, at the frequency corresponding to the secant period of the rehabilitated building. This secant period may be calculated by equating the maximum kinetic and strain energies in the building as follows:

$$T_s = 2\pi\sqrt{\frac{\sum_{i=1}^{N} w_i\delta_i^2}{g\sum_{i=1}^{N} F_i\delta_i}} \qquad \text{(C9-48)}$$

For an SDOF system, Equation C9-48 simplifies to:

$$T_s = 2\pi\sqrt{\frac{Dm}{V}} \qquad \text{(C9-49)}$$

where D is the displacement of the mass m, and V is the base shear corresponding to displacement D.

Analysis of Buildings Incorporating Displacement-Dependent Devices. Displacement-dependent energy dissipation devices should be explicitly represented in the mathematical model by bilinear, elasto-plastic, or rigid-plastic (friction) elements.

The Method 2 procedure for hysteretic energy dissipation devices is demonstrated below by the sample analysis of a one-story building for which friction devices are being considered.

Sample Analysis. For a one-story building, the push-over and spectral capacity curves are identical, namely,

- $N = 1$
- $\phi_{1m} = 1$
- $\delta_1 = \delta_r = D$
- $\Gamma_1 = 1$
- $W_{sm} = W_1$
- $S_a = Vg/W_1$
- $S_d = D$

The computed spectral capacity curves for the sample building (before and after rehabilitation) are shown in Figure C9-23, together with 20%, 30%, and 40% damped design demand curves.

The first step in the analysis procedure is to compute: (1) the force-displacement relation for the building before rehabilitation using push-over analysis, and (2) the effective damping in the building before rehabilitation (using Equation C9-45 and the force-displacement relation). The effective damping can be estimated using the bilinear hysteresis loop as follows.

The area contained within the hysteresis loop for the building is not precisely known, but is assumed to be a percentage of the area of the "perfect" bilinear hysteresis loop used to describe the computed push-over curve. For a bilinear system, where the spectral acceleration at lateral displacement D is defined as A, and the spectral acceleration at the yield displacement D_y is defined as A_y, the effective damping can be calculated as:

$$\beta_b = \frac{2(A_yD - D_yA)}{\pi AD} \qquad \text{(C9-50)}$$

The effective damping of the building is then computed as:

$$\beta_{eff} = q\beta_b \qquad \text{(C9-51)}$$

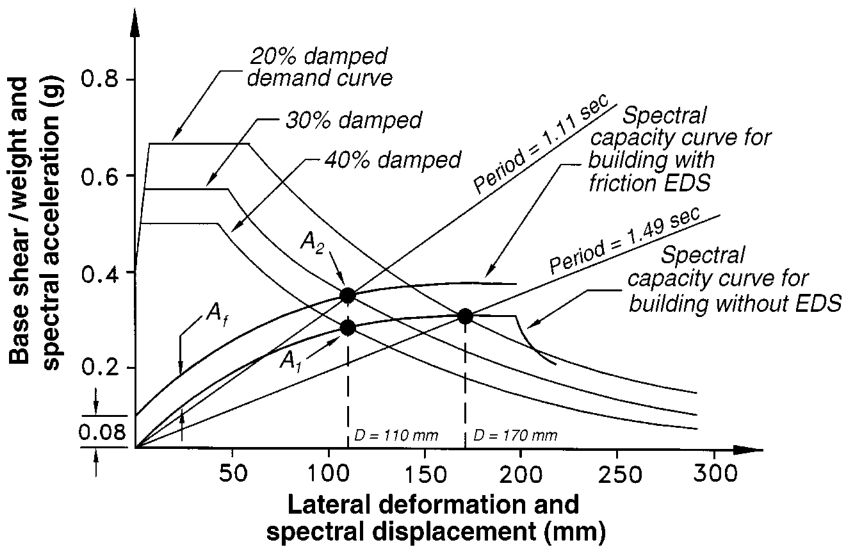


Figure C9-23 Spectral Capacity and Demand Curves for Rehabilitated One-Story Building

where q is a factor, less than one, equal to the ratio of the "actual" area of the hysteresis loop to that of the assumed perfect bilinear oscillator. Figure C9-24 shows the bilinear representation of the push-over curve, and the actual and perfect loop areas. For this example, q is approximately equal to 0.5. The value of factor q will depend on the type of construction and expected level of deformation. For example, a value $q = 0.2$ is inferred from the shake table test data of Li and Reinhorn (1995) for buildings rehabilitated with energy dissipation devices. Accordingly, it is recommended that a value of $q = 0.2$ be used for rehabilitated buildings unless a higher value can be justified.

The third step in the analysis procedure is to evaluate the spectral demand on the building before rehabilitation. The spectral demand is obtained iteratively by: (1) selecting points (displacements) on the spectral capacity curve—typically at the intersection of the spectral curve and the demand curves (e.g., 20%, 30%, and 40% damping); (2) calculating the effective damping of the building, β_{eff}, at the selected displacement points; and (3) comparing the calculated effective damping, β_{eff}, for each selected displacement point, with the demand curve damping value corresponding to the selected displacement point.

Returning to the sample building, consider the intersection point of the spectral capacity curve and the 20%-damped design demand curve at $D = 170$ mm,

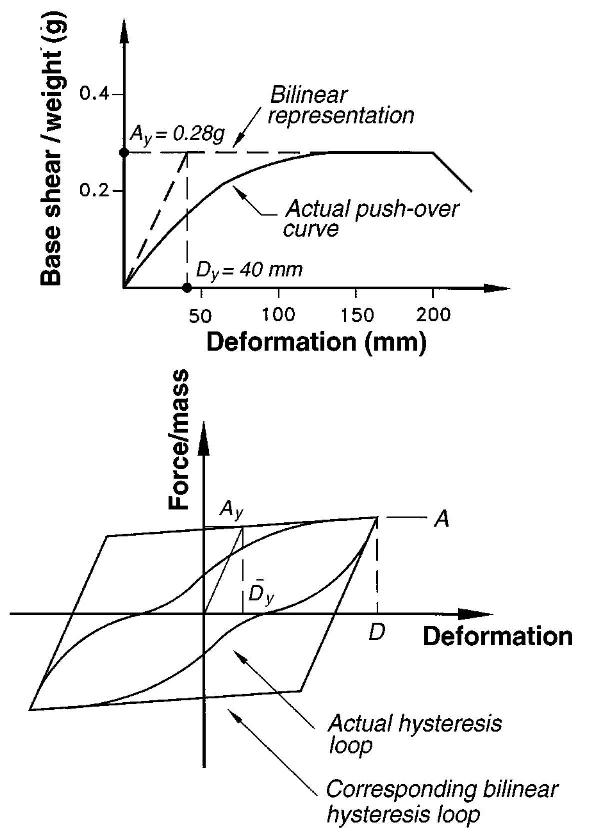


Figure C9-24 Representation of the Push-over Curve and Hysteresis Loops

$A = 0.31g$. Using values of $A_y = 0.28g$, $D_y = 40$ mm, and $q = 0.5$, the secant period T_s equals 1.5 seconds, β_b equals 0.40, and the effective damping of the building β_{eff} equals 0.20—the demand curve damping value associated with the trial displacement of 170 mm. No further iterations are necessary. The roof displacement demand on this one-story building before rehabilitation is therefore 170 mm (see Figure C9-23).

The three steps outlined above are repeated for the analysis of the rehabilitated building, as follows. The addition of friction energy dissipation devices serves to increase the strength of the sample building (as shown in Figure C9-23) by an amount assumed equal to A_f. The effective damping of the rehabilitated building is computed using Equation C9-45 by separating the hysteresis loop area into that area contributed by the energy dissipators (a near rectangular loop, if the energy dissipation device support framing is stiff), and the remainder of the rehabilitated building, as follows:

$$\beta_{eff} = \frac{2A_fD + 2q(A_yD - D_yA)}{\pi A_2 D} \qquad \text{(C9-52)}$$

where the spectral accelerations A_1 and A_2 are as defined in Figure C9-23. Following the procedure presented above, consider the intersection point of the spectral capacity curve for the rehabilitated building and the 30%-damped demand curve ($D = 110$ mm, $A_y = 0.28g$). Using $A_1 = 0.29g$, $A_2 = 0.36g$, $A_f = 0.08g$, $D_y = 40$ mm, and $q = 0.5$, the secant period T_s equals 1.11 seconds, and the effective damping β_{eff} is 0.30—the demand curve damping value associated with the trial displacement of 110 mm. No further iterations are necessary. The roof displacement demand on this one-story rehabilitated building is therefore 110 mm (see Figure C9-23)—65% of the displacement demand on the building before rehabilitation.

C. Analysis of Buildings Incorporating Velocity-Dependent Devices

Viscoelastic Energy Dissipation Devices. Viscoelastic energy dissipation devices exhibit effective stiffness that is generally dependent on frequency, amplitude of motion, and temperature. As such, the mathematical model of a rehabilitated building incorporating viscoelastic devices should account for the stiffness of these devices.

Viscoelastic devices should be modeled using linear or nonlinear springs representing the effective stiffness of the device at a fixed temperature and frequency. This frequency should be the inverse of the secant period of the structure with the added viscoelastic devices. The effect of temperature on the response of the viscoelastic device can be accounted for in the NSP by performing a series of analyses to bound the response of the rehabilitated building.

To demonstrate the analysis process, consider the sample one-story building with the friction devices replaced by viscoelastic devices. The displacement demand can be evaluated by calculating the effective damping:

$$\beta_{eff} = \frac{\frac{W_{DE}}{m} + 4q(A_yD - D_yA)}{2\pi A_2 D} \qquad \text{(C9-53)}$$

where m is the building mass, W_{DE} is the energy dissipated by the viscoelastic energy dissipation devices in one cycle of loading, and the remaining terms are as defined in Figure C9-25.

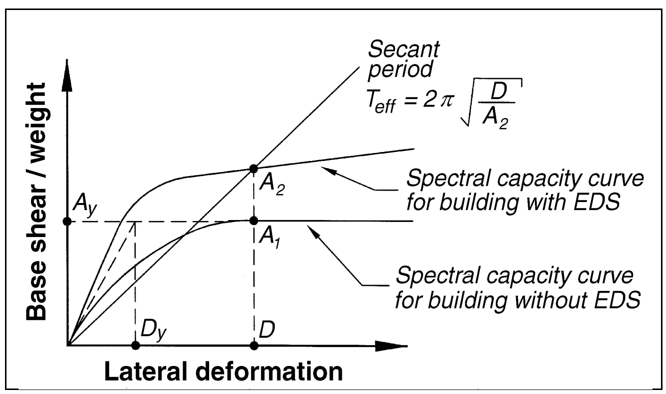


Figure C9-25 ***Definition of Parameters for Simplified Nonlinear Analysis of One-Story Building with Viscoelastic Energy Dissipation System (EDS)***

The energy dissipated by the viscoelastic energy dissipators can be calculated as:

$$W_{DE} = \frac{2\pi^2}{T_s}\sum_i C_j \cos^2\theta_j \Delta_{rj}^2 \qquad \text{(C9-54)}$$

where the summation extends over all energy dissipation devices; C_j is the damping coefficient of device j (Equations C9-27 and C9-28); θ_j is the angle of inclination of device j to the horizontal; and Δ_{rj} is the relative displacement of the attachment points of the energy dissipation device as shown in Figure C9-26.

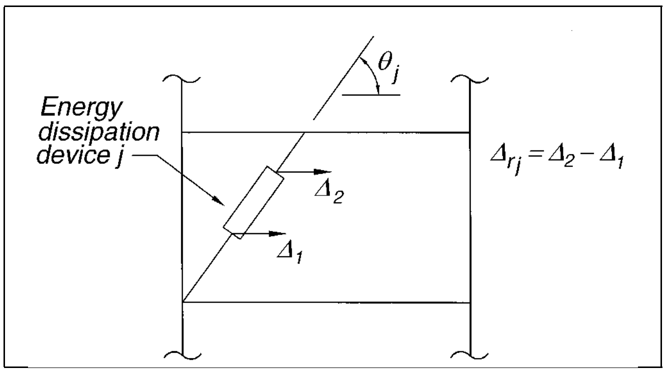


Figure C9-26 Definition of Angle and Relative Displacement of Energy Dissipation Device

The calculation of the capacity-demand intersection point follows the same procedure as that described above for displacement-dependent devices. For displacement-dependent devices, the member actions can be based on the forces and deformations associated with the capacity-demand intersection point. For velocity-dependent energy dissipation devices, one further step is needed to calculate member actions, because the calculated member forces are based solely on nodal displacements and do not include the member forces resulting from nodal velocities (or viscous forces). Separate analysis should be performed to quantify these effects using the peak viscous force along the axis of each viscoelastic energy dissipation device, calculated as follows:

$$F_j = \frac{2\pi}{T_s} C_j \Delta_{rj} \cos\theta_j \qquad \text{(C9-55)}$$

where C_j is the damping coefficient of device j at displacement amplitude $\Delta_{rj}\cos\theta_j$, and frequency equal to the inverse of the calculated secant period.

Fluid Viscous Energy Dissipation Devices. Fluid viscous energy dissipation devices do not generally exhibit stiffness. Accordingly, the push-over curve of the rehabilitated building, as determined by the NSP, is identical to that of the building without the energy dissipation system.

For a building with a capacity curve as shown in Figure C9-27, the effective damping is given by

$$\beta_{eff} = \frac{\dfrac{W_{DE}}{m} + 4q(A_y D - D_y A)}{2\pi A D} \qquad \text{(C9-56)}$$

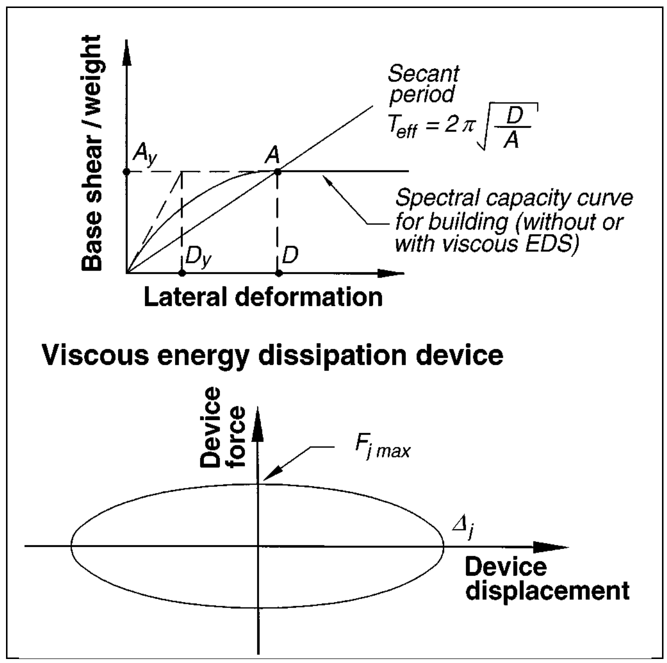


Figure C9-27 Definition of Parameters for Simplified Nonlinear Analysis of One-Story Building with Viscous Energy Dissipation System (EDS)

where W_{DE} is the work done by the viscous energy dissipation devices in one cycle of loading. For the general case of a nonlinear viscous device with force given by Equation C9-30, the work done (Soong and Constantinou, 1994) is:

$$W_{DE} = \sum_i \lambda F_{j\max} \Delta_j \qquad \text{(C9-57)}$$

where λ is a function of the velocity exponent as given in Table C9-4.

Table C9-4 Values of Parameter λ

Exponent α	Parameter λ
0.25	3.7
0.50	3.5
0.75	3.3
1.00	3.1
1.25	3.0
1.50	2.9
1.75	2.8
2.00	2.7

Alternatively, the work done may be expressed in terms of the relative displacement Δ_{rj} as defined in Figure C9-26:

$$W_{DE} = \left(\frac{2\pi}{T_s}\right)^{\alpha} \sum_i \lambda C_{0j} \left|\Delta_{rj} \cos\theta_j\right|^{1+\alpha} \qquad \text{(C9-58)}$$

where C_{0j} is the damping constant of device j (Equation C9-30). For a linear viscous device, for which the exponent α is equal to 1.0, Equation C9-58 takes the form:

$$W_{DE} = \frac{2\pi}{T_s} \sum_i C_{0j} \cos^2\theta_j \Delta_{rj}^2 \qquad \text{(C9-59)}$$

which is identical to Equation C9-54, except that C_{0j} is a constant in Equation C9-58, whereas C_j in Equation C9-54 is typically dependent on the excitation frequency and amplitude (velocity).

The calculation of the capacity-demand intersection point follows the same procedure as that described above for hysteretic and viscoelastic energy dissipation devices, except that Equations C9-56 through C9-59 are used to evaluate the effective damping of the rehabilitated building. Note that the push-over curve for the rehabilitated building will likely be different from that of the unrehabilitated building, because some existing framing elements are likely to require rehabilitation irrespective of the amount of damping added to the building. For displacement-dependent energy dissipation devices, the member actions can be based on the forces and deformations associated with the capacity-demand intersection point. For velocity-dependent energy dissipation devices, one further step is needed to calculate member actions, because the calculated member forces are based solely on nodal displacements and do not include the member forces resulting from nodal velocities (or viscous forces). Separate analysis should be performed to quantify these effects, using the peak viscous force along the axis of each viscous energy dissipation device calculated as follows:

$$F_j = \left(\frac{2\pi}{T_s}\right)^{\alpha} C_{0j} \left|\Delta_{rj} \cos\theta_j\right|^{\alpha} \qquad \text{(C9-60)}$$

where the secant period T_s is as defined in Figure C9-27.

A procedure to perform such an analysis is outlined in the discussion of Method 1 presented above.

The reader is referred to Section C9.3.9 for additional information on the implementation of energy dissipation devices using Method 2.

C9.3.5.2 Nonlinear Dynamic Procedure

If energy dissipation devices are dependent on loading 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 nonlinear Time-History Analysis. One means by which to account for variations in the force-deformation response of energy dissipation devices is to perform multiple analyses of the rehabilitated building, using the likely bounding response characteristics of the energy dissipation devices. The design of the rehabilitated building, including the energy dissipation devices, should be based on the maximum responses computed from the multiple analyses.

The viscous forces (if any) developed in the seismic framing system should be accounted for in the analysis and design of the seismic framing system. Evaluation of member action histories should be based on nodal displacements (operating on member stiffness matrices) and nodal velocities (operating on member damping matrices).

Key to the acceptable response of a rehabilitated building incorporating energy dissipation devices is the stable response of the energy dissipation devices. The forces and deformations in the energy dissipation devices that develop during the design earthquake should be demonstrated to be adequate by prototype testing per Section 9.3.8 of the *Guidelines*.

C9.3.6 Detailed Systems Requirements

C9.3.6.1 General

No commentary is provided for this section.

C9.3.6.2 Operating Temperature

No commentary is provided for this section.

C9.3.6.3 Environmental Conditions

Energy dissipation devices should be designed with consideration given to environmental conditions, including aging effects, creep, fatigue, ambient temperature, and exposure to moisture and damaging substances. Although such considerations are unusual for conventional construction materials, the key role played by the energy dissipation devices makes it imperative that the environment in which the devices will be installed be considered carefully in the design process.

C9.3.6.4 Wind Forces

Rehabilitated buildings incorporating energy dissipation devices that are subject to failure by low-cycle fatigue (e.g., steel-yielding dampers) should resist the prescribed design wind forces in the elastic range to avoid premature failure.

Other devices that incorporate seals for containing fluids should be investigated for the possibility of seal malfunction and fluid loss, which could result in a substantial reduction of the energy dissipation capability of the device.

Wind-induced displacements in velocity-dependent devices may provide temperature increase in the device that may be of significance and require special considerations in the design of the device.

C9.3.6.5 Inspection and Replacement

Unlike conventional construction materials that are inspected infrequently—or never—some types of energy dissipation hardware will require regular inspection. Further, post-installation testing of certain types of hardware may be prudent, given the limited data available on the aging characteristics of the innovative materials and fluids being proposed for energy dissipation devices. Accordingly, easy access for both routine inspection and testing and scheduled or earthquake-mandated replacement of energy dissipation devices should be provided.

C9.3.6.6 Manufacturing Quality Control

Key to the acceptable response of a building rehabilitated using energy dissipation devices is the reliable response of those devices. Such reliance on the response of the energy dissipation devices makes necessary the implementation of a rigorous production quality control testing program.

C9.3.6.7 Maintenance

Such energy dissipation devices as friction dampers, fluid viscous dampers, viscoelastic dampers, and other mechanical dampers may require periodic maintenance and testing. Devices based on metallic-yielding and the plastic flow of lead likely need no 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.

C9.3.7 Design and Construction Review

C9.3.7.1 General

Design and construction issues associated with the use of energy dissipation devices are not well understood by many design professionals, due primarily to the limited use of this emerging technology at the time of this writing. Accordingly, all phases of the design and construction of buildings rehabilitated with energy dissipation devices should be reviewed by an independent engineering review panel. This panel should include persons experienced in seismic analysis and the theory and application of energy dissipation devices.

The peer review should commence during the preliminary design phase of the rehabilitation project and continue through the installation of the energy dissipation devices.

C9.3.8 Required Tests of Energy Dissipation Devices

C9.3.8.1 General

No commentary is provided for this section.

C9.3.8.2 Prototype Tests

A. General

Although reduced-scale prototypes are permitted for certain tests described in Section 9.3.8.1, full-scale tests should be specified wherever possible. Failure characteristics of devices should not be determined by reduced-scale testing.

B. Data Recording

At least one hundred data points per cycle of testing should be recorded to capture the force-displacement response of the device adequately.

C. Sequence and Cycles of Testing

Prototype testing of energy dissipation devices is necessary to confirm the assumptions made in the analysis and design of the rehabilitated building, and to demonstrate that the energy dissipation hardware can sustain multiple cycles of deformation associated with the design wind storm, and the BSE-2.

At least one full-size energy dissipation device of each predominant type and size to be used in the rehabilitated building should be tested. These prototype devices should be fabricated using the identical material and processes proposed for the fabrication of the production devices.

Each prototype energy dissipation device should generally be subjected to a minimum of 2,000 displacement cycles of an amplitude equal to that expected in the design wind storm. The goals of this test are twofold, namely, (1) to demonstrate that the fatigue life of the device will not be exhausted in the design wind storm, and (2) to provide the engineer of record with design properties for the device in the design wind storm. For short-period buildings, the devices may see more than 2,000 significant displacement cycles in the design wind storm; for such buildings, the number of displacement cycles should be increased.

D. Devices Dependent on Velocity and/or Frequency of Excitation

Given the key role played by energy dissipation devices, it is appropriate that these devices be exhaustively tested. The testing program presented in the *Guidelines* is limited in scope and warrants augmentation on a project-by-project basis. As a minimum, each prototype device should be subjected to 20 displacement cycles corresponding to the BSE-2; the frequency of testing should be representative of the frequency characteristics of the building for the BSE-2.

The rules given in the *Guidelines* for evaluating frequency dependence are based on similar rules developed for testing base isolators. The frequency range of 0.5 f_1 to 2.0 f_1 should bound the frequency response of a building. The frequency of 2.0 f_1 corresponds to a stiffer building than that assumed in design (perhaps due to nonstructural components); the frequency of 0.5 f_1 corresponds to a fourfold decrease in building stiffness due to the effects of earthquake shaking—likely an upper bound for a rehabilitated building. Data from these tests should fall within the limiting values assumed by the engineer of record for the design of the building.

E. Devices Dependent on Bilateral Displacement

If the force-displacement properties of an energy dissipation device are influenced by building displacements in the direction perpendicular to the longitudinal axis of the energy dissipation device (termed bilateral displacement), such influence should be investigated by testing. The force-displacement response of the prototype device should be recorded at two levels of bilateral displacement: zero displacement, and the displacement equal to that calculated in the design earthquake. Data from these tests should fall within the limiting values assumed by the engineer of record for the design of the building.

F. Testing Similar Devices

No commentary is provided for this section.

C9.3.8.3 Determination of Force-Displacement Characteristics

The force-deformation characteristics of an energy dissipation device should be assessed using the cyclic test results of Section 9.3.8.2. The equations given for effective stiffness (k_{eff}) and effective damping (β_{eff}) are strictly valid only for viscoelastic devices.

C9.3.8.4 System Adequacy

Given the use of multiple Performance Levels in the *Guidelines*, the engineer of record may choose to augment the prototype testing requirements with tests at displacement levels different from those specified. These additional tests would serve to confirm the assumptions made in the analysis regarding the response of the energy dissipation devices at varying levels of building response.

C9.3.9 Example Applications of Analysis Procedures

C9.3.9.1 Introduction

The purpose of this section is to demonstrate by example some of the procedures presented in Section 9.3 of the *Guidelines*. Specifically, the use of the Linear Static, Linear Dynamic, and Nonlinear Static Procedures are described in Sections C9.3.9.3, C9.3.9.4, and C9.3.9.5, respectively.

The sample building used in this study is composed of a series of three-story, three-bay frames (see Figure C9-28). The effects of torsion are ignored and two-dimensional analysis is used for evaluation. The tributary floor weights are shown in Figure C9-28. For clarity, the frame is modeled as shear-type building with the story shear-story drift relations shown in Figure C9-28. The solution of the eigen problem for this frame results in the modal data presented in Table C9-5.

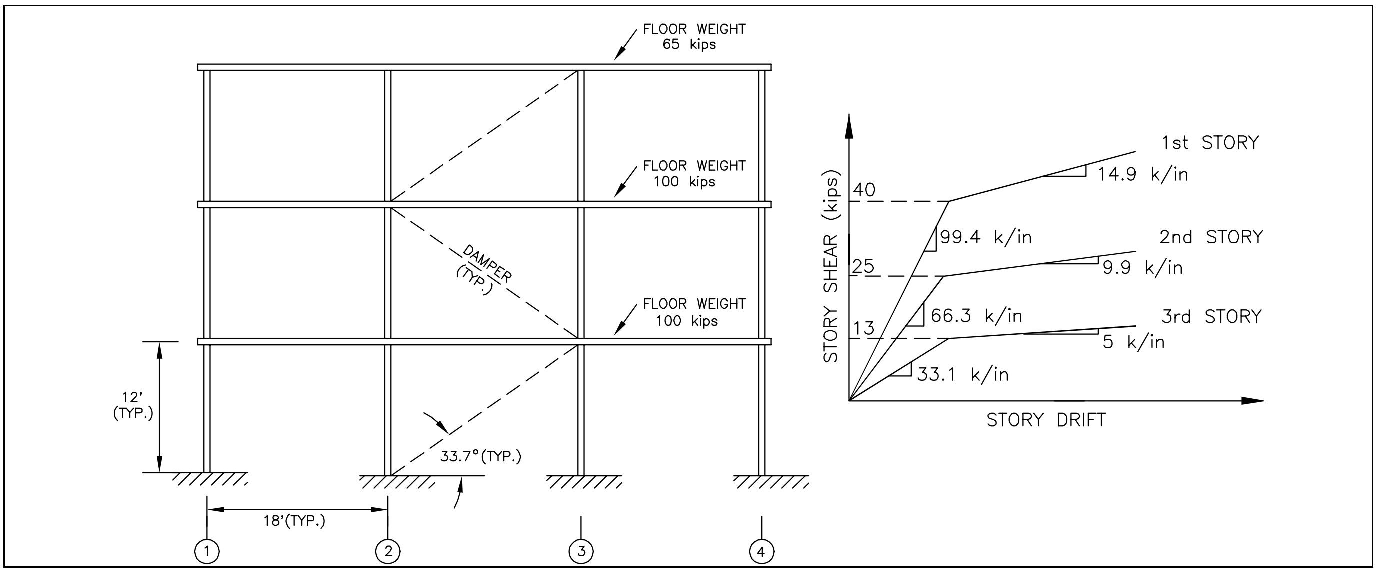


Figure C9-28 Sample Building Information

For the purpose of this study, the energy dissipation devices are assumed to be linear viscous dampers. (No preference for such dampers is inferred by this assumption.) Further, the mechanical characteristics of the sample dampers are assumed to be independent of excitation frequency, bilateral displacement, and ambient and operating temperature. (However, this may not be a reasonable assumption and must be investigated by the engineer as a key part of the design process.) The energy dissipation system consists of three linear fluid viscous dampers located in the central bay of the building as shown in Figure C9-28. It is assumed that all three dampers have identical properties (damping coefficient) and that the properties are to be selected to provide damping for the linear procedure of 20% in the fundamental mode. Assuming 5% damping in the building frame, the effective damping of the building is 25% of critical (see Equation 9-28). The

Table C9-5 Modal Analysis of the Sample Building Using Elastic Properties

	Mode 1	Mode 2	Mode 3	Reference
Period (sec)	0.75	0.34	0.22	
Frequency (rad/s)	8.38	18.45	28.46	
Mode Shapes				
Roof	1	1	1	
2	0.64	–0.73	–3.10	
1	0.29	–0.62	4.67	
Modal Weight (kips)	218.3	31.3	15.3	Equation C9-39
Participation Factor	1.38	0.45	0.07	
Effective Damping	0.25	0.67	0.63	
Coefficient B_s or B_1	2.05	3.0	3.0	Table 2-15
Spectral Accel. (g)	0.49	0.33	0.33	Spectral demand divided by appropriate *B*
Spectral Displ. (in)	2.69	0.38	0.16	
Factor CF_1	0.89	0.60	0.62	Equation 9-31
Factor CF_2	0.45	0.80	0.78	Equation 9-32

braced framing supporting the dampers is initially assumed to be infinitely rigid. This assumption is investigated further later in this section.

The seismic hazard at the site of the sample building is described by the 5%-damped response spectrum of Figure C9-22, with S_{DS} = 1, S_{D1} = 0.6, and T_0 = 0.6 second.

C9.3.9.2 Properties of Energy Dissipation Devices

The damping coefficient for each damper is selected to provide 20% of critical damping in the fundamental mode using elastic component properties. Using the eigen data presented in Table C9-5—β equal to 0.05, β_{eff} equal to 0.25, and θ_j equal to 33.7° at all three levels—the calculated value for C_0 is 4.28 kip-sec/in.

C9.3.9.3 Application of the Linear Static Procedure (LSP)

Analysis of the building using the LSP is permitted, provided the building frame remains elastic, the effective damping in the fundamental mode is less than 30% of critical, and criteria regarding the maximum resistance of the energy dissipation devices are satisfied (see item 1 in Section 9.3.4.1B).

A. Pseudo Lateral Load

The pseudo lateral load for the LSP is calculated using Equation 3-6. For the sample building, $C_1 = C_2 = 1.0$ for a building responding in the elastic range, $C_3 = 1.0$ if second-order effects are ignored, T = 0.75 second from the eigen analysis, W = 265 kips, β_{eff} = 0.25, B_s = 2.05 (Table 2-15) and B_1 = 1.6 (Table 2-15). The cutoff period for the modified spectrum (= $T_0 B_s/B_1$) is 0.77 second. The fundamental period of the building is less than the cutoff period. The spectral acceleration can therefore be calculated as equal to:

$$S_a = \frac{S_{DS}}{B_s} = \frac{1.0}{2.05} = 0.49g \qquad \text{(C9-61)}$$

and the pseudo lateral load is equal to 129 kips.

B. Vertical Distribution of Seismic Force

The vertical distribution of the pseudo lateral load V is calculated using Equation 3-8. The exponent k is equal to 1.12 for T equal to 0.75 second. The vertical distribution factors are equal to:

$$C_{v3} = 0.41$$

$$C_{v2} = 0.40$$

$$C_{v1} = 0.19$$

The lateral loads are calculated as the product of the vertical distribution factors and *V*. These loads represent the inertial forces at the time of maximum displacement.

C. Linear Analysis Results

The member forces at the time of maximum displacement are calculated by routine analysis using the story inertial forces calculated above. The relative axial displacements in the dampers can be calculated as the product of the inter-story drift and the cosine of the angle of inclination of the dampers to the horizontal plane (= 33.7 degrees in this instance for all three stories).

At the time of maximum velocity, the damper relative axial velocities are calculated as the product of the damper relative axial displacement at the time of maximum displacement, the damping coefficient (C_0), and the first modal radial frequency (= 8.38 radians per second).

State combination factors CF_1 and CF_2 are calculated to determine component actions at the time of maximum acceleration. Using Equations 9-31 and 9-32, the state factors are calculated to be equal to 0.89 and 0.45, respectively.

Table C9-6 summarizes key story shear data. Figure C9-29 shows the forces acting on the frame at the three stages identified above. Actions in one first story column are shown. The capacity of the column should be checked for all three sets of actions.

The limit on the use of the LSP set forth in item 1 of Section 9.3.4.1B can be evaluated using the data presented in Table C9-6. The maximum resistance of the frame, exclusive of the energy dissipation devices, is calculated as the resistance at maximum displacement in the BSE-2. Assume that the specified seismic hazard is that associated with the BSE-2. The resistances of each story of the frame at the maximum displacement are listed in the last column in Table C9-6. The maximum resistance of the energy dissipation devices in each story is equal to the horizontal component of the maximum damper axial forces: 40 kips, 39 kips, and 32 kips, in the third, second, and first stories, respectively. The criterion of item 1 is therefore violated and the design must be modified.

As an aside, consider the third column in the third story. The gravity load carried by this column is approximately 22 kips (based on tributary areas). The maximum axial load delivered by the damper is 27 kips—producing a maximum compression load of 49 kips and a maximum tension load of 5 kips.

Table C9-6 Summary of Results of the LSP

Floor or Story	Lateral Load	Floor Displ.	Story Drift	Damper Axial Displ. (in.)	Damper Axial Veloc. (in./s)	Damper Axial Force (kips)	Story Shear at Maximum Drift (kips)
3	53.4	4.504	1.613	1.342	11.243	48.1*	53.4
2	52.0	2.891	1.590	1.323	11.082	47.4	105.4
1	23.9	1.301	1.301	1.082	9.068	38.8	129.3

***** Horizontal component exceeds 50% of story shear at maximum drift.

D. Damper Support Framing

To maximize the effect of the supplemental damping hardware, the damper support should be stiff so as to maximize the relative displacement and velocity between the ends of the damper. Assuming that more than four dampers are installed in the sample building in each principal direction and in each story, and that the dampers are installed in line with the bracing, the braces must be designed for a minimum axial force equal to 130% of the maximum axial force in the damper. For the brace supporting the third story damper, the minimum design axial force is equal to 62.5 kips (= 1.3

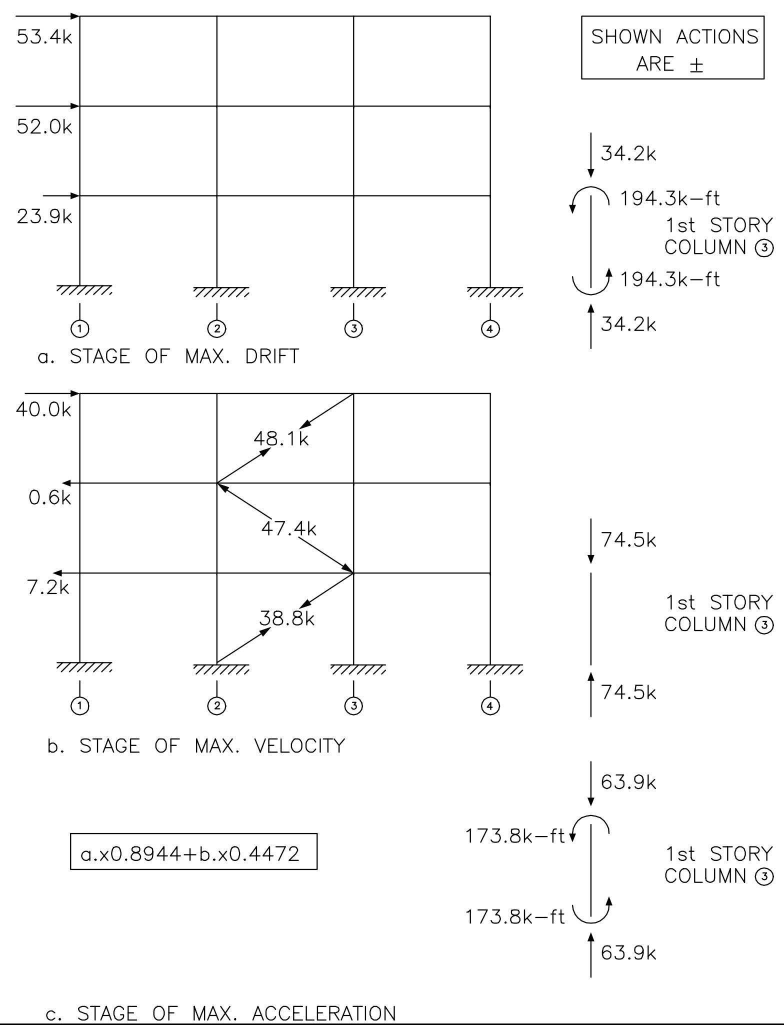


Figure C9-29 Loads on Building and LSP Actions in a Selected Component

x 48.1). Strength design can be used to design the brace without additional load factors. A 6 in. x 6 in. x 0.25 in. tube section (F_y = 46 ksi) is sufficient for this purpose. The stiffness of this brace (K_b) is 625 kips/inch. The brace-damper system can be idealized as a spring-dashpot system (Maxwell model)—see Figure C9-21. This spring-dashpot system has stiffness K' and damping coefficient C given by Constantinou et al. (1996):

$$K' = \frac{C_0 \tau \omega^2}{1 + \tau^2 \omega^2}, \quad C = \frac{C_0}{1 + \tau^2 \omega^2}, \quad \tau = \frac{C_0}{K_b} \quad \text{(C9-62)}$$

where ω is the circular frequency (= 8.38 radians/sec.). Substituting C_0 equal to 4.28 k-sec/inch into Equation C9-62 produces stiffness equal to 2.1 kips/inch and a damping coefficient equal to 4.27 k-sec/inch.

The calculated stiffness K' of 2.1 kips/inch is small by comparison with the minimum story stiffness of 33.1 kips/inch and will not appreciably alter the dynamic characteristics of the frame. Further, the damping coefficient is essentially unchanged. Accordingly, analysis based on the assumption of infinite brace stiffness is most adequate for this example.

C9.3.9.4 Application of the Linear Dynamic Procedure (LDP)

The sample frame and energy dissipation devices studied in Section C9.3.9.3 are analyzed using the response spectrum method. Calculations are performed for each of the three modes. Table C9-5 presents modal properties and Table C9-7 presents calculated modal responses and modal responses combined by the SRSS rule. Figure C9-30 presents the forces in the frame at the times of maximum displacement, velocity, and drift. Actions in a selected first story column are presented at the bottom of Figure C9-30. The capacity of this column should be checked for all three sets of actions and the actions due to the SRSS combination.

C9.3.9.5 Application of the Nonlinear Static Procedure (NSP)

One NSP is presented in the *Guidelines* (Method 1). Two procedures are described in this *Commentary* (Method 1 and Method 2). The two methods differ only in the means by which the roof displacement is determined. In Method 1, the target roof displacement is given by Equation 3-11. In Method 2, the roof displacement is calculated by comparison of the spectral capacity curve and design demand spectrum (see Section C9.3.5.1); Figure C9-31 illustrates the steps in Method 2 that are described in Section C9.3.5.1. The two methods should produce similar results unless the strength ratio R (see Equation 3-12) is greater than 5. For buildings with small strength ratios, the NDP is recommended.

Table C9-7 Summary of Results of the LDP

Response Quantity	Floor/Story	Mode 1	Mode 2	Mode 3	SRSS
Floor Displacement (in.)	3	3.70	0.17	0.01	3.70
	2	2.38	–0.12	–0.03	2.39
	1	1.07	–0.11	0.05	1.08
Story Drift (in.)	3	1.32	0.29	0.04	1.34
	2	1.31	0.02	0.08	1.32
	1	1.07	0.11	0.05	1.08
Damper Axial Displacement (in)	3	1.10	0.24	0.04	1.12
	2	1.09	0.02	0.07	1.09
	1	0.89	0.09	0.04	0.90
Damper Axial Velocity (inches/sec.)	3	9.194	4.484	1.065	10.284
	2	9.152	0.276	2.012	9.375
	1	7.472	1.612	1.207	7.739
Damper Axial Force (kips)	3	39.3	19.2	4.6	44.0
	2	39.2	1.2	8.6	40.2
	1	32.0	6.9	5.2	33.1
Story Shear at Maximum Drift (kips)	3	43.7	9.7	1.5	44.8
	2	87.1	1.2	5.6	87.3
	1	106.6	10.4	5.1	107.2
Inertia Force at Maximum Drift (kips)	3	43.7	9.7	1.5	
	2	43.4	–10.9	–7.1	
	1	19.5	–9.2	10.7	

A. Force-Displacement Relations

Evaluation of the relationships between base shear force and roof displacement is key to the NSP. For the sample building, the mathematical model is subjected to two load patterns: (1) loads proportional to floor weights (uniform pattern), and (2) loads proportion to the vertical distribution factors of Equation 3-8 (modal pattern). The force-displacement relations (also termed push-over curves) for these two load patterns are shown in Figure C9-32. The force-displacement relations are evaluated to displacements greater than the target displacement. At a minimum, the relation should be established for control node displacements equal to 150% of the target displacement.

For the sample building, the effective stiffness at 60% of the yield displacement is equal to the initial stiffness

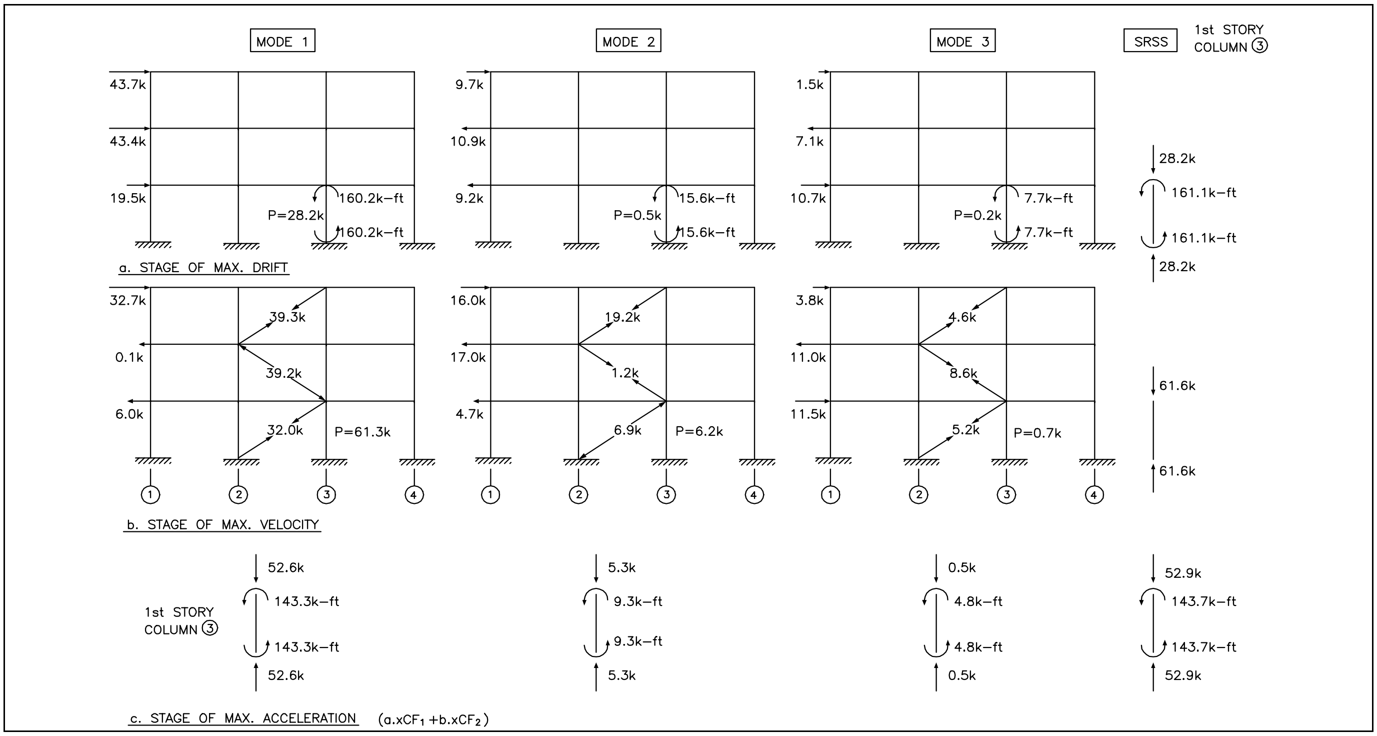


Figure C9-30 Loads on Building and LDP Actions in a Selected Component

and the effective period (T_e) is equal to 0.75 second (see Table C9-5).

Table C9-8 lists the modal properties of the building at different levels of roof displacement calculated using the modal load pattern. For this calculation, member stiffnesses are modified by the ratio of the secant stiffness at the selected displacement level to the effective elastic stiffness. For comparison, the elastic modal properties, appropriate for roof displacements less than 1.1 inches, are given in Table C9-5. Although modal periods increase with increasing roof displacements, the modal shapes and participation factors are somewhat invariant to changes in stiffness.

Table C9-9 presents modal data corresponding to the use of a uniform load pattern. A comparison of the modal data presented in Tables C9-8 and C9-9, at identical levels of roof displacement, demonstrates why multiple load patterns must be considered. Namely, substantially different modal properties may be obtained if different load patterns are used.

B. Fundamental Mode Response Estimates, Method 2, Modal Pattern

The analysis is performed first using the modal pattern of loads. An initial roof displacement of 4.2 inches is assumed. Equations C9-43 and C9-44 are used to convert the force-displacement relation (push-over curve) to the corresponding spectral capacity curve. Modal properties at the roof displacement of 4.2 inches are used for this purpose. A bilinear representation of the spectral capacity curve is shown in Figure C9-33a.

The effective damping is calculated by Equation 9-36. The damping afforded by the building frame, exclusive of the dampers, may either be assumed to be equal to 0.05 or determined using Equation C9-51 as follows. Values for D and A are calculated at the assumed roof displacement of 4.2 inches: D = 3.05 inches and A = 0.24 g; factor q is assumed to be equal to 0.2. The calculated damping in the frame, exclusive of the dampers, is 0.055. The damping ratio provided by the energy dissipators of 0.37 is calculated using Equations 9-36 and 9-37 and the modal properties corresponding to a roof displacement of 4.2 inches (equal to the assumed roof displacement). The effective damping in the rehabilitated building is 0.42 (= 0.05 + 0.37).

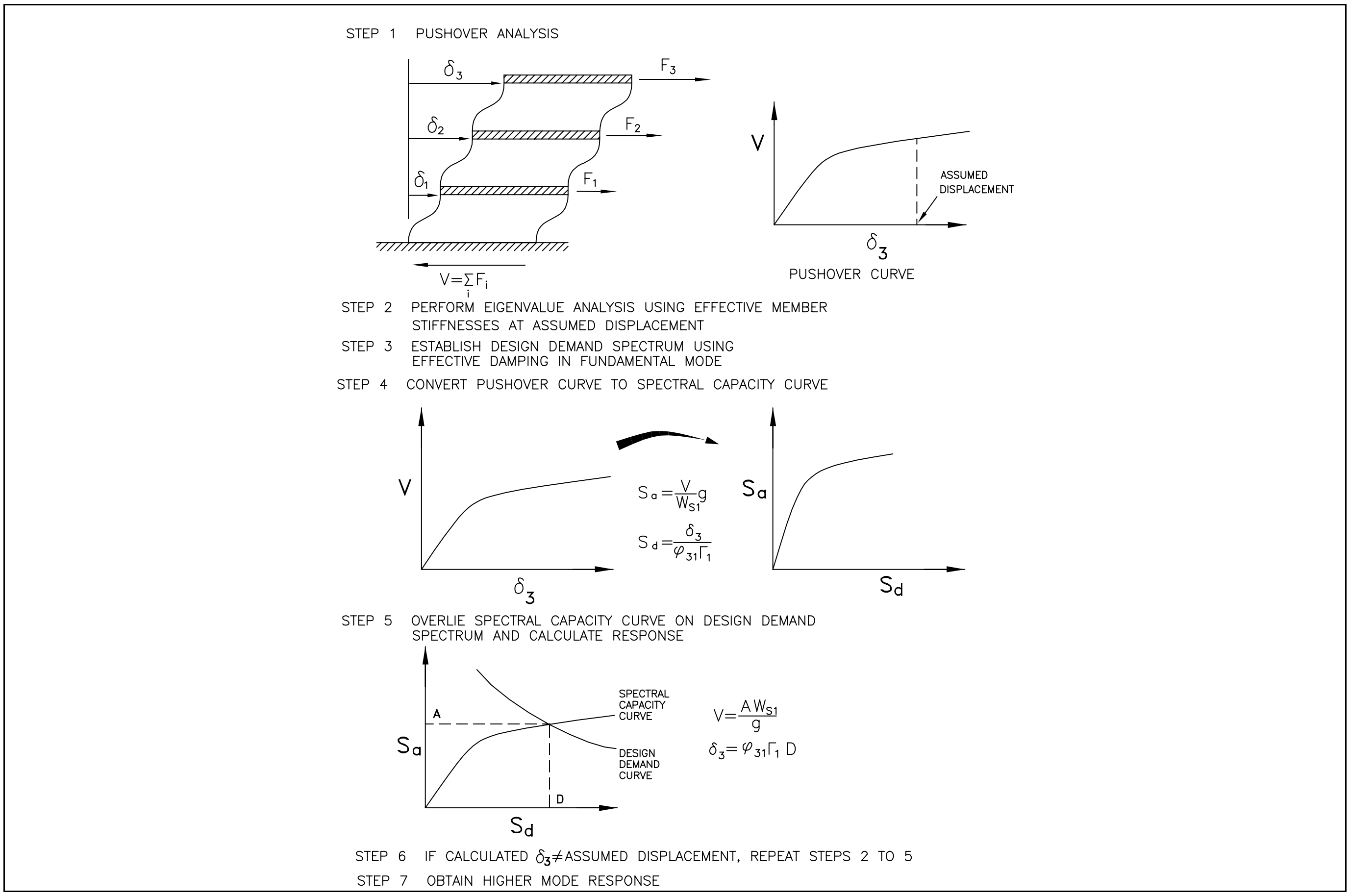


Figure C9-31 NSP Method 2 Schematic

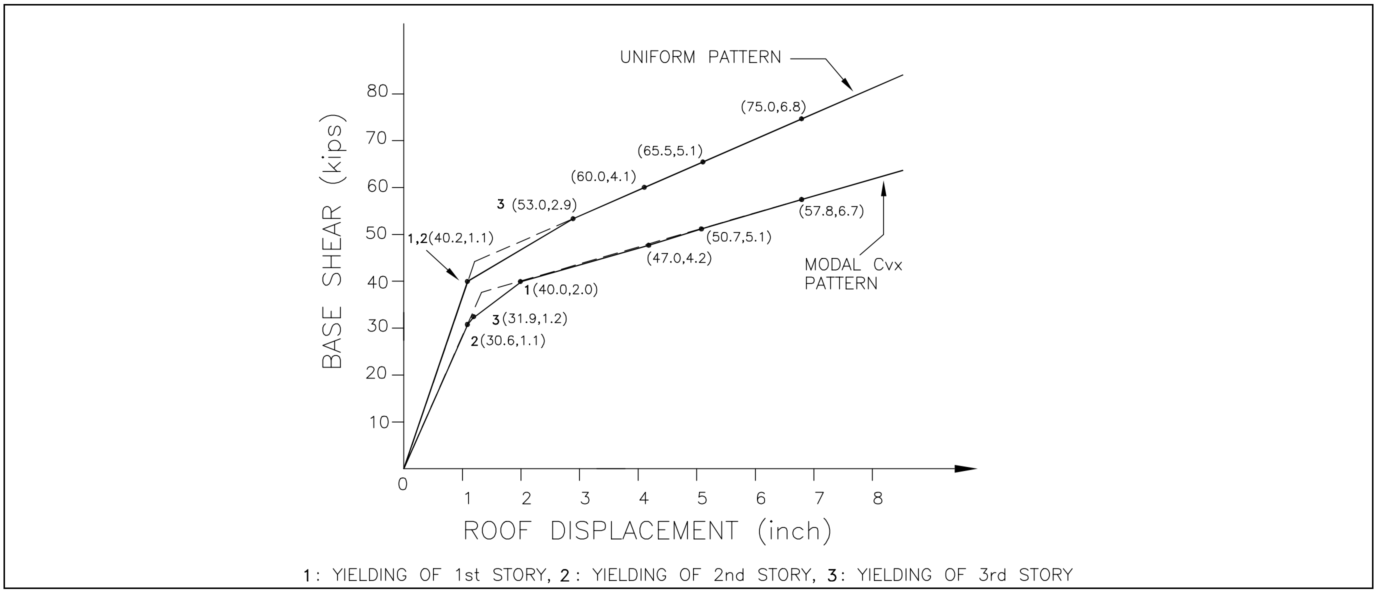


Figure C9-32 Force-Displacement Relations for Sample Building

The design demand curve is established using the 5%-damped spectrum modified to reflect the effective damping in the building. For effective damping equal to 0.42, and a first mode period of 1.19 second, the damping modification factor (for T equal to 1.19 second) is equal to 1.92. The resulting design demand curve is presented in Figure C9-33a. The intersection of the design demand and spectrum capacity curves (D = 3.7 inches, A = 0.26 g) corresponds to the target displacement. This information is converted to base shear and roof displacement using Equations C9-40 and C9-41, resulting in a base shear force equal to 50.7 kips and a roof displacement of 5.1 inches.

Table C9-8 Displacement-Dependent Modal Properties, Modal Load Pattern

Roof Displacement	Parameter	Mode 1	Mode 2	Mode 3
4.2 inches	T_i (sec.)	1.19	0.54	0.35
	ω (rad./sec)	5.28	11.59	18.21
	Mode shape ordinates (θ_i)	1	1	1
		0.60	–0.92	–3.75
		0.19	–0.49	8.29
	W_{si} (kips)	199	34	32
	Γ_i	1.38	0.44	0.06
5.1 inches	T_i (sec.)	1.26	0.57	0.37
	ω (rad./sec)	5.00	10.97	17.07
	Mode shape ordinates (θ_i)	1	1	1
		0.61	–0.89	–3.58
		0.21	–0.52	7.30
	W_{si} (kips)	203	34	28
	Γ_i	1.38	0.44	0.07
6.1 inches	T_i (sec.)	1.32	0.60	0.39
	ω (rad./sec)	4.75	10.44	16.15
	Mode shape ordinates (θ_i)	1	1	1
		0.62	–0.86	–3.45
		0.23	–0.55	6.62
	W_{si} (kips)	205	34	26
	Γ_i	1.38	0.45	0.07

The calculated roof displacement of 5.1 inches is not equal to the assumed displacement of 4.2 inches. The procedure outlined above is repeated using an assumed roof displacement of 5.1 inches and modal properties corresponding to this displacement (see Table C9-8). The updated spectral capacity curve is shown in Figure C9-33b. The revised effective damping is equal to 0.44 (= 0.05 + 0.39); the damping modification factor corresponding to the revised effective damping ratio is equal to 1.94.

The updated design demand curve is shown in Figure C9-33b. The intersection point of the design demand and spectrum capacity curves is (D = 3.7 inches, A = 0.25 g). The corresponding roof displacement and base shear force are 5.1 inches and 50.6 kips, respectively. The calculated and assumed roof displacements are equal and no further iterations are required.

The floor displacements and story drifts in the first mode are those calculated at the roof displacement of

Table C9-9 Displacement-Dependent Modal Properties, Uniform Load Pattern

Roof Displacement	Parameter	Mode 1	Mode 2	Mode 3
4.2 inches	T_i (sec.)	1.22	0.48	0.36
	ω (rad./sec)	5.14	13.15	17.50
	Mode shape ordinates (θ_i)	1	1	1
		0.78	–0.46	–1.58
		0.35	–0.83	1.64
	W_{si} (kips)	230	26	9
	Γ_i	1.29	0.41	0.12
5.1 inches	T_i (sec.)	1.30	0.53	0.39
	ω (rad./sec)	4.82	11.93	16.30
	Mode shape ordinates (θ_i)	1	1	1
		0.75	–0.52	–1.84
		0.34	–0.75	2.16
	W_{si} (kips)	228	26	11
	Γ_i	1.31	0.42	0.11
6.1 inches	T_i (sec.)	1.39	0.57	0.41
	ω (rad./sec)	4.52	11.06	15.39
	Mode shape ordinates (θ_i)	1	1	1
		0.75	–0.52	–1.94
		0.36	–0.74	2.24
	W_{si} (kips)	230	25	10
	Γ_i	1.31	0.41	0.10

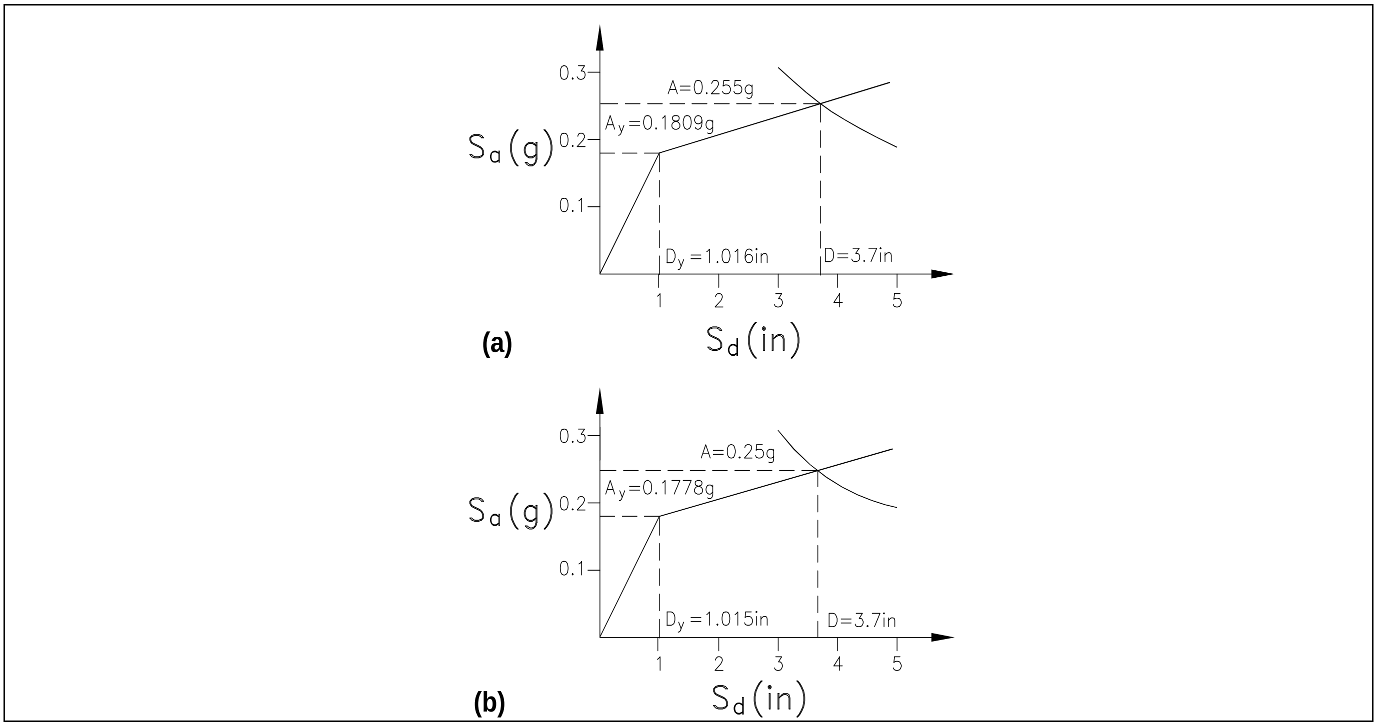


Figure C9-33 NSP Response Estimates, Method 2, Modal Pattern (a) Target Roof Displacement of 4.2 inches (b) Target Roof Displacement of 5.1 inches

5.1 inches. See Table C9-10 for details. Axial displacements and forces in the energy dissipation devices are also presented in this table. These data were calculated using the first modal frequency calculated at the roof displacement of 5.1 inches (= 5.00 radians/sec from Table C9-8).

C. Higher Mode Response Estimates, Method 2, Modal Pattern

Higher mode responses are evaluated using the Response Spectrum Method. The modal properties corresponding to a roof displacement of 5.1 inches are used. The effective modal damping is calculated using Equation 9-33 and estimates of the modal frequencies and modal displacements (see also Equation 9-30). The calculated effective modal damping ratios are 1.21 and 0.97 in the second and third modes, respectively. Near-critical damping presents a complication because conventional modal analysis can no longer be applied. However, given the upper limit on the value of B_S or B_1 (equal to 3.0 below the transition point in the spectrum), and recognizing that the stated spectrum reduction method generally produces conservative estimates of displacement and velocity (Constantinou et al., 1996), the procedure outlined above is acceptable for highly-damped systems. Note that the maximum acceleration in the short-period range for highly-damped systems will be approximately equal to the peak ground acceleration.

Table C9-10 Summary of Results of the NSP, Method 2, Modal Pattern

Response Quantity	Floor/Story	Mode 1	Mode 2	Mode 3	SRSS
Lateral Loads (kips)	3	20.9	9.6	1.4	
	2	20.4	–13.1	–7.8	
	1	9.3	–7.8	16.0	
Floor Displacement (inches)	3	5.11	0.47	0.03	5.13
	2	3.14	–0.42	–0.10	3.17
	1	1.11	–0.25	0.21	1.16
Story Drift (inches)	3	1.97	0.90	0.13	2.17
	2	2.02	0.17	0.32	2.05
	1	1.11	0.25	0.21	1.16
Damper Axial Displacement (inches)	3	1.64	0.75	0.11	1.81
	2	1.68	0.14	0.26	1.71
	1	0.93	0.21	0.18	0.97
Damper Axial Force (kips)	3	35.1	35.0	8.1	50.2
	2	36.0	6.7	19.1	41.3
	1	19.8	9.7	12.8	25.5
Actions in First Story Column 3 (P: kips, M: k-ft)	Maximum Drift	P = 13.6 M = 76.0	P = 0 M = 17.0	P = 0 M = 14.3	P = 13.6 M = 79.2
	Maximum Velocity	P = 50.4 M = 0.0	P = 10.3 M = 0.0	P = 1.0 M = 0.0	P = 51.5 M = 0.0
	Maximum Acceleration	P = 43.6 M = 56.9	P = 10.3 M = 17.0	P = 1.0 M = 14.3	P = 44.8 M = 61.1

Higher mode responses are calculated using a damping modification factor of 3.0 and with combination factors CF_1 and CF_2 both equal to 1.0. The latter assumption is conservative but likely appropriate for highly-damped modes. Higher mode response data are presented in columns 4 and 5 of Table C9-10. Total responses calculated using the SRSS modal combination rule are presented in column 6 of the table.

Consider the data presented in this table. It is evident that mode 1 displacement response is dominant; for design purposes, higher mode displacements can generally be ignored. However, the same argument cannot be made when considering the maximum forces in the dampers. Of particular interest is the damper axial force in the first story. The mode 3 damper force is more than 60% of the mode 1 damper force. Clearly, higher mode effects must be evaluated when designing viscous dampers, damper support framing, and columns to which viscous forces can be delivered.

D. Response Estimates, Method 2, Uniform Pattern

The procedure used to evaluate the response of the sample building is identical to that outlined above except that the modal properties are established using a uniform load pattern (see Table C9-9). Starting with an assumed roof displacement of 5.1 inches, the first iteration produces a calculated roof displacement of 4.84 inches (within 5% of the assumed value). Given that modal properties are not significantly affected by displacement (see Table C9-9), no further iterations are required. Modal actions and deformations are calculated using the same procedure as that outlined above. Responses are summarized in Table C9-11.

C9.4 Other Response Control Systems

Base isolation (Section 9.2) and passive energy dissipation (Section 9.3) systems are seismic response control systems. When included in a rehabilitated building, these systems generally reduce inertia forces and drifts during earthquake shaking, thereby reducing or eliminating damage. These systems achieve this objective by either deflecting a portion of the seismic energy (base isolation) or converting kinetic energy in the framing system to heat (energy dissipation).

Other response control systems, designed and implemented for nonseismic applications, are being further developed for seismic applications. Two such classes of control systems are dynamic vibration absorbers and active control systems.

Table C9-11 Summary of Results of the NSP, Method 2, Uniform Pattern

Response Quantity	Floor/Story	Mode 1	Mode 2	Mode 3	SRSS
Lateral Loads (kips)	3 2 1	15.7 24.2 24.2	9.1 –7.3 –10.5	2.4 –6.8 8.0	
Floor Displacement (inches)	3 2 1	4.84 3.90 2.02	0.38 –0.20 –0.29	0.05 –0.10 0.12	4.85 3.91 2.04
Story Drift (inches)	3 2 1	0.94 1.88 2.02	0.58 0.09 0.29	0.15 0.22 0.12	1.11 1.90 2.04
Damper Axial Displacement (inches)	3 2 1	0.78 1.57 1.68	0.48 0.07 0.24	0.13 0.18 0.10	0.92 1.58 1.70
Damper Axial Force (kips)	3 2 1	16.0 32.3 34.7	24.6 3.6 12.1	8.9 12.5 6.7	30.7 34.8 37.4
Actions in First Story Column 3 (P: kips, M: k-ft)	Maximum Drift	P = 13.5 M = 96.2	P = 1 M = 13.1	P = 0 M = 5.4	P = 13.6 M = 97.2
	Maximum Velocity	P = 46.1 M = 0.0	P = 8.9 M = 0.0	P = 1.7 M = 0.0	P = 47.0 M = 0.0
	Maximum Acceleration	P = 38.1 M = 77.7	P = 9.9 M = 13.1	P = 1.7 M = 5.4	P = 39.4 M = 79.0

C9.4.1 Dynamic Vibration Absorbers

Dynamic vibration absorbers are oscillators that, when properly tuned and attached to a framing system, transfer kinetic energy among the vibrating modes, leading to an increase in damping in the selected mode of vibration (Den Hartog, 1956). Examples of these absorbers are tuned mass dampers (TMDs) and tuned liquid dampers (TLDs). The reader is referred to International Association for Structural Control (1994), and Soong and Constantinou (1994) for additional information.

Tuned mass dampers consist of a mass, a restoring force (spring, viscoelastic material, or pendulum action), and a means of dissipating energy (viscous damper, viscoelastic material, or friction). When attached at a point of significant vibration, and tuned to a frequency close to the fundamental frequency of the framing system, TMDs produce a combined structure-appendage system with increased damping.

Tuned liquid dampers may take one of the following forms: (1) a tuned sloshing damper in which liquid (typically water) in a large container serves as the tuned mass, with damping resulting from either fluid sloshing or fluid flow through screens; or (2) a tuned liquid column damper that utilizes the vibration of a liquid in a U-shaped container, inducing damping by restricting the flow of the fluid through an orifice (Sakai, 1989; Kareem, 1994; Soong and Constantinou, 1994). TLDs are tuned by selecting the proper dimensions of the fluid containers; however, the frequency and damping characteristics of a TLD may be motion-dependent, that is, nonlinear.

Dynamic vibration absorbers have been used to reduce the response of structures to wind excitation, occupant activity, and machine vibration. In buildings, their use has been restricted to enhancing comfort for the occupants of tall buildings. Moreover, their application has been restricted to structures that remain in the elastic range, so that tuning is maintained during dynamic excitation. The effectiveness of a dynamic vibration absorber is significantly reduced when the structural system undergoes significant inelastic action (Kaynia et al., 1981; Sladek and Klingner, 1983), although studies summarized in Villaverde (1994) indicate that with the use of massive and highly damped vibration absorbers, it is possible to control the seismically-induced response of structures.

To date, the use of dynamic vibration absorption hardware to control the seismic response of buildings in severe earthquakes has not been demonstrated. Research and studies in this field are ongoing.

C9.4.2 Active Control Systems

The subject of active seismic control is broad. The reader is referred to Soong (1990), Soong and Constantinou (1994), ATC (1993), and International Association for Structural Control (1994) for detailed information on both active control theory and active control applications.

Active control systems are based on the premise that it is possible to modify the dynamic behavior of a structural system by the use of an automated control system composed of sensors, controllers, and actuators. The sensors measure the response of the structure. The controller processes the signals from the sensors, computes the required control forces based on a control algorithm, and supplies control signals to the actuators. The actuators impose the computed forces or displacements on the building.

To understand the function of an active control system, it is worthwhile to review the function of a passive control system, the elements of which are shown in Figure C9-34. The energy dissipation system is an integral part of the structure and develops motion control forces. The power needed to generate these forces is provided by the motion of the framing system during dynamic excitation; the amplitude and direction of these forces are based entirely on the relative motion of the attachment points of the energy dissipation devices.

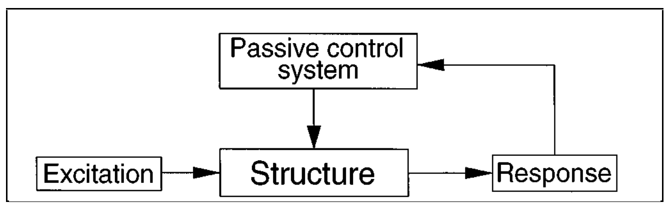


Figure C9-34 Elements of Passive Control System

An active control system also develops motion control forces, as illustrated in Figure C9-35. The sample active control system shown in Figure C9-36 is an active bracing system in which hydraulic actuators serve as the active braces (Reinhorn et al., 1992). The magnitude and direction of these forces are determined by the controller, which receives information on the response of the structure from the strategically located sensors.

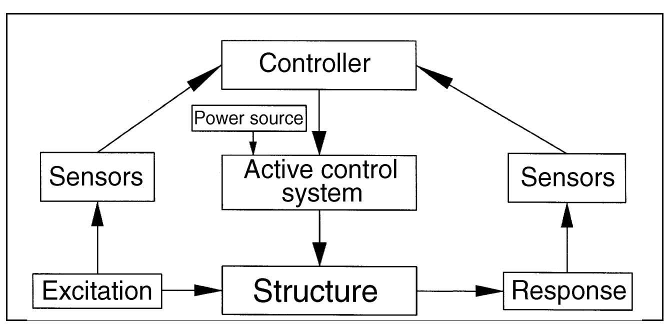


Figure C9-35 Elements of Active Control System

In principle, an active control system should provide better response control than a passive control system. However, the effective operation of active control systems is currently hampered by two significant shortcomings. First, the control forces required for mitigating the effects of strong seismic excitations are so large that the control system and its power source may assume a prohibitively large size. Second, active control systems are highly sophisticated, require continuous maintenance, and have not yet reached the level of reliability required for seismic applications. Accordingly, active control systems have not yet been used for seismic applications.

Research in active control continues at a pace that almost assures the development of practical active control systems for seismic applications in the near future. An example of new developments in this field is that of "semi-active" control systems. The term "semi-active" denotes that the operation of the control system consumes only a small amount of external power. In a semi-active control system, the mechanical properties of the system are continuously updated using sensor-based feedback from the framing system (as in active control systems), and the motion of the building is used to develop the control forces (as in passive control systems) necessary to adjust the damping and/or stiffness characteristics of the semi-active control system. Further, because the control forces in a semi-active system always oppose the motion of the building, the system is inherently more stable than an active control system. Semi-active control systems are typically considered to be fail-safe, in that the semi-active energy dissipation devices can be designed to exhibit prescribed damping and stiffness characteristics in the event of a complete loss of power (Patten et al., 1993; Symans et al., 1994). Figure C9-37 shows the elements of a sample semi-active energy dissipating bracing system. In this system, semi-active energy dissipators are used as bracing members. A direct-drive servovalve is used to adjust the damping coefficient of the semi-active brace. In the event of a loss of power, this servovalve is designed to close, upon which the semi-active energy dissipating braces convert to passive energy dissipating braces with a high-damping coefficient. An alternative use of semi-active devices is described in Liang et al. (1995).

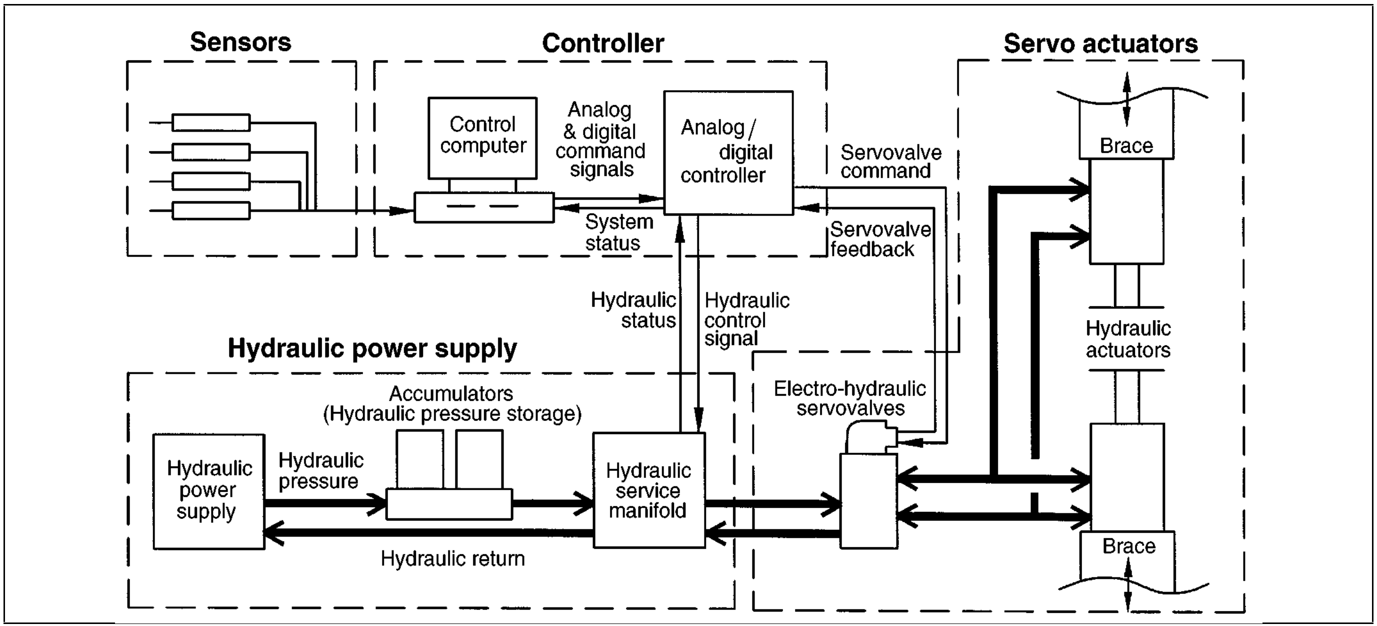


Figure C9-36 Details of Control System of Active Bracing System

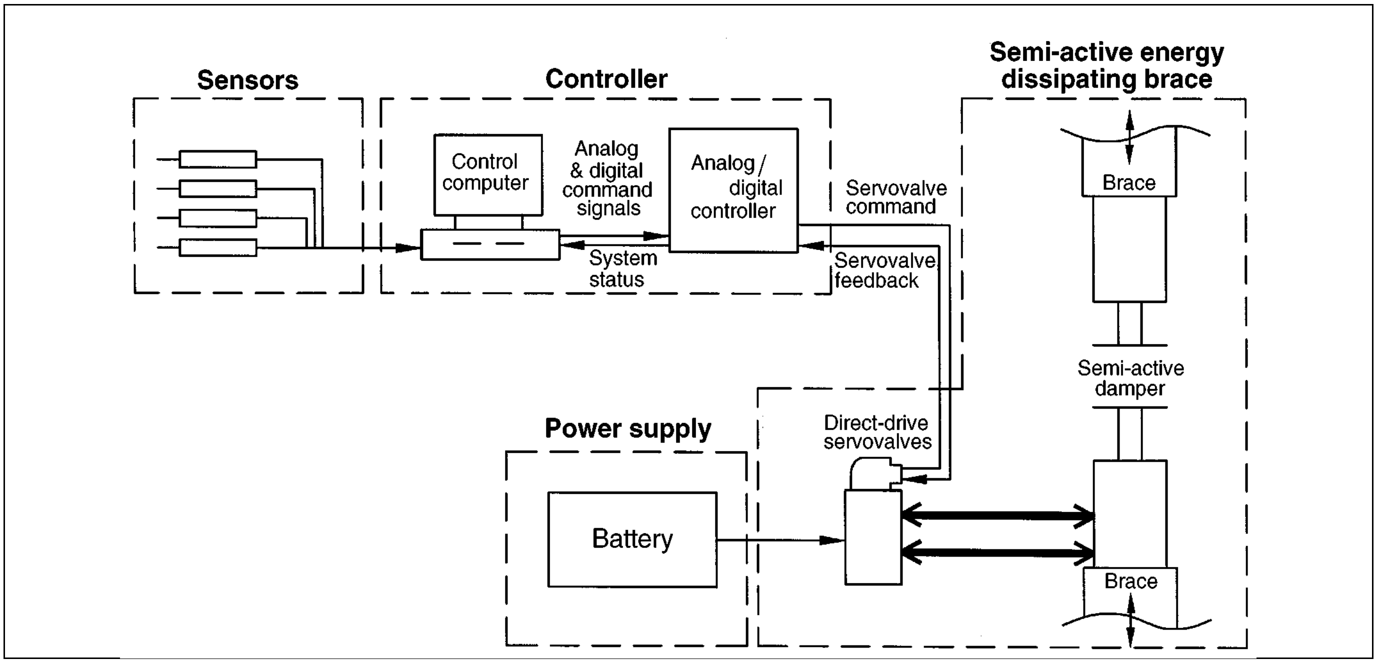


Figure C9-37 Details of Control System of Semi-Active Energy Dissipation Bracing System

C9.5 Definitions

Push-over curve: The base shear versus roof displacement relationship computed using the Nonlinear Static Procedure of Chapter 3.

Spectral capacity curve: The spectral acceleration versus spectral displacement relationship based on the capacity push-over curve as described in Section 9.3.

C9.6 Symbols

No commentary is provided for this section.

C9.7 References

Aiken, I. D., and Kelly, J. M., 1990, *Earthquake Simulator Testing and Analytical Studies of Two Energy-Absorbing Systems for Multistory Structures*, Report No. EERC-90/03, Earthquake Engineering Research Center, University of California, Berkeley, California.

Aiken, I. D., Nims, D. K., Whittaker, A. S., and Kelly, J. M., 1993, “Testing of Passive Energy Dissipation Systems,” *Earthquake Spectra*, Earthquake Engineering Research Institute, Oakland, California, Vol. 9, No. 3, pp. 335–370.

Al-Hussaini, T., Zayas, V., and Constantinou, M. C., 1994, *Seismic Isolation of Multi-Story Frame Structures using Spherical Sliding Isolation Systems*, Report No. NCEER-94-0007, National Center for Earthquake Engineering Research, State University of New York at Buffalo, New York.

Amin, N., Mokha, A., and Fatehi, H., 1993, “Seismic isolation retrofit of the U.S. Court of Appeals Building,” *Proceedings of Seminar on Seismic Isolation, Passive Energy Dissipation, and Active Control*, Applied Technology Council Report No. ATC-17-1, Redwood City, California.

ASTM, latest edition, Standard D4014, American Society of Testing Materials, Philadelphia, Pennsylvania.

ATC, 1993, *Proceedings of Seminar on Seismic Isolation, Passive Energy Dissipation, and Active Control*, Report No. ATC-17-1, Applied Technology Council, Redwood City, California.

Bergman, D. M., and Hanson, R. D., 1993, "Viscoelastic Mechanical Damping Devices at Real Earthquake Displacements," *Earthquake Spectra*, Earthquake Engineering Research Institute, Oakland, California, Vol. 9, No. 3, pp. 389–412.

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.

Chang, K. C., Soong, T. T., Oh, S.-T., and Lai, M. L., 1991, *Seismic Response of a 2/5 Scale Steel Structure with Added Viscoelastic Dampers*, Report No. NCEER-91-0012, National Center for Earthquake Engineering Research, State University of New York at Buffalo, New York.

Cho, D. M., and Retamal, E., 1993, "The Los Angeles County Emergency Operations Center on High-Damping Rubber Bearings to Withstand an Earthquake Bigger than the Big One," *Proceedings of Seminar on Seismic Isolation, Passive Energy Dissipation, and Active Control*, Applied Technology Council Report No. ATC-17-1, Redwood City, California, pp. 209–220.

Chopra, A. K., 1995, *Dynamics of Structures*, Prentice-Hall, Englewood Cliffs, New Jersey.

CSI, 1994, *ETABS (Version 6.0): Linear and Nonlinear, Static and Dynamic Analysis and Design of Building Systems*, Computers and Structures, Inc., Berkeley, California.

Constantinou, M. C., Soong, T. T., and Dargush, G. F., 1996, *Passive Energy Dissipation Systems for Structural Design and Retrofit*, National Center for Earthquake Engineering Research, State University of New York at Buffalo, New York.

Constantinou, M. C., and Symans, M. D., 1993, "Experimental Study of Seismic Response of Buildings with Supplemental Fluid Dampers," *The Structural Design of Tall Buildings*, John Wiley & Sons, New York, New York, Vol. 2, pp. 93–132.

Constantinou, M. C., Tsopelas, P. C., Kim, Y.-S., and Okamoto, S., 1993, *NCEER-TAISEI Corporation Research Program on Sliding Seismic Isolation Systems for Bridges: Experimental and Analytical Study of Friction Pendulum System (FPS)*, Report No. NCEER-93-0020, National Center for Earthquake Engineering, State University of New York at Buffalo, New York.

Den Hartog, J. P., 1956, *Mechanical Vibrations*, Dover Publications, New York, New York.

EERI, 1993, "Theme Issue: Passive Energy Dissipation," *Earthquake Spectra*, Earthquake Engineering Research Institute, Oakland, California, Vol. 9, No. 3.

Grigorian, C. E., and Popov, E. P., 1994, *Energy Dissipation with Slotted Bolted Connections*, Report No. UCB/EERC-94/02, Earthquake Engineering Research Center, University of California, Berkeley, California.

Hart, G. G., et al., 1990, "Seismic Strengthening of a Tall Building Incorporating Base Isolation," *Proceedings: Fourth U.S. National Conference on Earthquake Engineering*, Earthquake Engineering Research Institute, Oakland, California.

Honeck, W., Walters, M., Sattary, V., and Rodler, P., 1993, "The Seismic Isolation of Oakland City Hall," *Proceedings of Seminar on Seismic Isolation, Passive Energy Dissipation, and Active Control*, Applied Technology Council Report No. ATC-17-1, Redwood City, California.

International Association for Structural Control, 1994, *Proceedings of First World Conference on Structural Control*, Los Angeles, California.

ICBO, 1991, "Division III—Earthquake Regulations for Seismic-Isolated Structures," Chapter 23, *Uniform Building Code*, 1991 Edition, International Conference of Building Officials, Whittier, California.

ICBO, 1994, "Division III—Earthquake Regulations for Seismic-Isolated Structures," Chapter 16, *Uniform Building Code*, 1991 Edition, International Conference of Building Officials, Whittier, California.

ICBO, 1995, "SEAOC Seismology Committee Code Change Proposal" for Chapter 16, Division III (Isolation Provisions) of the 1997 *UBC*, International Conference of Building Officials, Whittier, California.

Kareem, A., 1994, "The Next Generation of Tuned Liquid Dampers," *Proceedings of First World*

Conference on Structural Control, Los Angeles, CA, pp. FP5-19 to FP5-28.

Kasai, K., Munshi, J. A., Lai, M. L., and Maison, B. F., 1993, "Viscoelastic Damper Hysteretic Model: Theory, Experiment, and Application," *Proceedings: Seminar on Seismic Isolation, Passive Energy Dissipation, and Active Control*, Applied Technology Council Report No. ATC-17-1, Redwood City, California.

Kaynia, A. M., Veneziano, D., and Biggs, J. M., 1981, "Seismic Effectiveness of Tuned Mass Dampers," *Journal of the Structural Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 107, pp. 1465–1484.

Kelly, J. M., 1988, *Base Isolation in Japan, 1988*, Report No. UCB/EERC-88/20, Earthquake Engineering Research Center, University of California, Berkeley, California.

Kelly, J. M., 1993, *Earthquake-Resistant Design with Rubber*, Springer-Verlag, London, United Kingdom.

Kircher, C. A., and Bachman, R. E., 1991, "Guidelines for Design Criteria for Base Isolation Retrofit of Existing Buildings," *Proceedings, 60th Annual Convention*, Structural Engineers Association of California, Sacramento, California.

Li, C., and Reinhorn, A. M., 1995, *Experimental Study and Analytical Investigation of Seismic Retrofit of Structures with Supplemental Damping: Part II—Friction Devices*, Report No. NCEER-95-0009, National Center for Earthquake Engineering Research, State University of New York at Buffalo, New York.

Liang, Z., Tong, M., and Lee, G. C., 1995, *Real Time Structural Parameter Modification (RSPM): Development of Innervated Structures*, Report No. NCEER-95-0012, National Center for Earthquake Engineering Research, State University of New York at Buffalo, New York.

Makris, N., Constantinou, M. C., and Dargush, G. F., 1993, "Analytical Model of Viscoelastic Fluid Dampers," *Journal of the Structural Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 119, No. 11, pp. 3310–3325.

Mayes, R. M., 1988, "Analysis, Design and Testing of the Isolation System for the Salt Lake City & County Building," *International Seismic Isolation/Historic Preservation Symposium, Salt Lake City*, Salt Lake City Corporation, Salt Lake City, Utah.

Murota, N., Goda, K., Suzusi, S., Sudo, C., and Suizu, Y., 1994, "Recovery Characteristics of Dynamic Properties of High-Damping Rubber Bearings," *Proceedings of Third U.S.–Japan Workshop on Earthquake Protective Systems for Bridges, Berkeley, California, 1994*, Report No. NCEER 94-0009, National Center for Earthquake Engineering Research, State University of New York at Buffalo, New York, pp. 1-63 to 2-76.

Naaseh, Simin, 1995, "Seismic Retrofit of San Francisco City Hall - The Role of Masonry and Concrete," *Proceedings of the Third National Concrete and Masonry Engineering Conference*, San Francisco, California.

Nagarajaiah, S., Reinhorn, A., and Constantinou, M. C., 1991, *3D-BASIS: Nonlinear Dynamic Analysis of Three Dimensional Base Isolate Structures*, Report No. NCEER-91-0005, National Center for Earthquake Engineering Research, State University of New York at Buffalo, New York.

Newmark, N. M. and Hall, W. J., 1982, *Earthquake Spectra and Design*, Earthquake Engineering Research Institute, Oakland, California.

Nims, D. F., Richter, P. J., and Bachman, R. E., 1993, "The Use of the Energy Dissipation Restraint for Seismic Hazard Mitigation," *Earthquake Spectra*, Earthquake Engineering Research Institute, Oakland, California, Vol. 9, No. 3, pp. 467–498.

Patten, W. N., Sack, R. L., Yen, W., Mo, C., and Wug, H. C., 1993, "Seismic Motion Control Using Semi-active Hydraulic Force Actuators," *Proceedings of Seminar On Isolation, Passive Energy Dissipation, and Active Control*, Applied Technology Council Report No. ATC-17-1, Redwood City, California, pp. 727–736.

Pekcan, G., Mander, J., and Chen, S., 1995, "The Seismic Response of a 1:3 Scale Model RC Structure with Elastomeric Spring Dampers," *Earthquake Spectra*, Earthquake Engineering Research Institute, Oakland, California, Vol. 11, No. 2, pp. 249–267.

Pyle, S. L., Janseen, A. G., Holmes, W. T., and Kircher, C. A., 1993, "Life-Cycle Cost Study for the State of

California Justice Building," *Proceedings of Seminar on Seismic Isolation, Passive Energy Dissipation, and Active Control*, Applied Technology Council Report No. ATC-17-1, Redwood City, California.

Reinhorn, A. M., Li, C., and Constantinou, M. C., 1995, *Experimental and Analytical Investigation of Seismic Retrofit of Structures with Supplemental Damping Part I: Fluid Viscous Damping Devices*, Report No. NCEER-95-0001, National Center for Earthquake Engineering Research, State University of New York at Buffalo, New York.

Reinhorn, A. M., Nagarajaiah, S., Constantinou, M. C., Tsopelas, P., and Li, R., 1994, *3D-BASIS-TABS (Version 2.0): Computer Program for Nonlinear Dynamic Analysis of Three-Dimensional Base Isolated Structures*, Report No. NCEER-94-0018, National Center for Earthquake Engineering Research, State University of New York at Buffalo, New York.

Reinhorn, A. M., Soong, T. T., Lin, R. C., Riley, M. A., Wang, Y. P., Aizawa, S., and Higashino, M., 1992, *Active Bracing System: A Full Scale Implementation of Active Control*, Report No. NCEER-92-0020, National Center for Earthquake Engineering Research, State University of New York at Buffalo, New York.

Sakai, F., 1989, "Tuned Liquid Column Damper - New Type Device for Suppression of Building Vibrations," *Proceedings, International Conference on High Rise Buildings*, Nanjing, China.

SEAOC, 1986, *Tentative Seismic Isolation Design Requirements*, Structural Engineers Association of California, San Francisco, California.

SEAOC, 1990, *Recommended Lateral Force Requirements and Commentary*, Fifth Edition, Seismology Committee, Structural Engineers Association of California, Sacramento, California.

SEAOC, 1996, *Recommended Lateral Force Requirements and Commentary*, Sixth Edition, Seismology Committee, Structural Engineers Association of California, Sacramento, California.

Skinner, R. I., Robinson, W. H., and McVerry, G. H., 1993, *An Introduction to Seismic Isolation*, J. Wiley & Sons, New York, New York.

Sladek, J. R., and Klingner, R.E., 1983, "Effect of Tuned-Mass Dampers on Seismic Response," *Journal of the Structural Engineering Division*, American Society of Civil Engineers, New York, New York, No. 109, pp. 2004–2009.

Soong, T. T., 1990, *Active Structural Control: Theory and Practice*, Longman, London, United Kingdom.

Soong, T. T., and Constantinou, M. C., 1994, *Passive and Active Structural Vibration Control in Civil Engineering*, Springer-Verlag, Wien-New York.

Symans, M. D., Constantinou, M. C., Taylor, D. P., and Garnjost, K. D., 1994, "Semi-Active Fluid Dampers for Seismic Response Control," *Proceedings of First World Conference on Structural Control*, Los Angeles, California, pp. FA4-3 to FA4-12.

Tsopelas, P., and Constantinou, M. C., 1994, *Experimental and Analytical Study of Systems Consisting of Sliding Bearings and Fluid Restoring Force-Damping Devices*, Report No. NCEER 94-0010, National Center for Earthquake Engineering Research, State University of New York at Buffalo, New York.

Tsopelas, P. C., Constantinou, M. C., and Reinhorn, A. M., 1994, *3D-BASIS-ME Computer Program for Nonlinear Dynamic Analysis of Seismically Isolated Single and Multiple Structures and Liquid Storage Tanks*, Report No. NCEER-94-0014, National Center for Earthquake Engineering Research, State University of New York at Buffalo, New York.

Villaverde, R., 1994, "Seismic Control of Structures with Damped Resonant Appendages," *Proceedings of First World Conference on Structural Control*, Los Angeles, California, pp. WP4-113 to WP4-122.

Way, D., and Howard, J., 1990, "Seismic Rehabilitation of the Mackay School of Mines, Phase III, with Base Isolation," *Earthquake Spectra*, Earthquake Engineering Research Institute, Oakland, California, Vol. 6, No. 2.

Whittaker, A. S., Bertero, V., Alonso, J., and Thompson, C., 1989, *Earthquake Simulator Testing of Steel Plate Added Damping and Stiffness Elements*, Report No. EERC-89/02, Earthquake Engineering Research Center, University of California, Berkeley, California.

Winters, C. W., and Constantinou, M. C., 1993, *Evaluation of Static and Response Spectrum Analysis Procedures of SEAOC/UBC for Seismic Isolated Structure*s, Report No. NCEER-93-0004, National Center for Earthquake Engineering Research, State University of New York at Buffalo, New York.

Yang, T.-S., and Popov, E. P., 1995, *Experimental and Analytical Studies of Steel Connections and Energy Dissipators,* Report No. UCB/EERC-95/13, Earthquake Engineering Research Center, University of California, Berkeley, California.

Zayas, V. A., Low, S. S., and Mahin, S. A., 1987, *The FPS Earthquake Resisting System: Experimental Report*, Report No. UCB/EERC-87/01, Earthquake Engineering Research Center, University of California, Berkeley, California.

Zayas, V., and Low, S. S., 1991, "Steel Seismic Isolators Applied to a Wood Frame Building," *Proceedings, 60th Annual Convention,* Structural Engineers Association of California, Sacramento, California.

C10. Simplified Rehabilitation

C10.1 Scope

FEMA 178, *NEHRP Handbook for the Seismic Evaluation of Existing Buildings* (BSSC, 1992a), following the lead of *ATC-14* (ATC, 1987) and *ATC-22* (ATC, 1989), catalogued expected seismic performance by defining Model Building Types in terms of generalized structural systems, loads, load paths, and potential weaknesses. The result of an application of the FEMA 178 evaluation method is the determination that a building either meets its safety criteria, or rehabilitation is needed to correct specific deficiencies. The potential weaknesses used in FEMA 178 were identified using the past behavior of building types, and presented as evaluation statements to be answered "true" or "false," with appropriate procedures suggested for detailed evaluation when necessary. For a particular building, each statement that has a false answer flags a potential area for concern and subsequent analysis. In this manner, the evaluating engineer is led through a consideration and evaluation of the entire structural system to the point of determining whether a building meets the FEMA 178 (BSSC, 1992a) life safety standard, which includes structural and nonstructural criteria based on a specified probability of ground motion. The Simplified Rehabilitation Method is based on the deficiencies defined in FEMA 178 (BSSC, 1992a) and the simple concept that elimination of each deficiency is sufficient for rehabilitation.

The FEMA 178 process and the model buildings presented therein are the basis for the Model Building Types used in this chapter. FEMA 178 (BSSC, 1992a) defined 15 Model Building Types, described in Table 10-2 of the *Guidelines*, that were developed to represent all typical styles of construction throughout the United States. This categorization of buildings has been used throughout the FEMA guideline series and is used here for consistency.

Since these models were first introduced in 1987, however, it has become evident that there were more styles of construction for several classes of buildings. The differences were generally found in the type of diaphragm system, either flexible—in the case of wood or untopped metal deck—or stiff—as in concrete or metal deck with concrete fill. It was decided that, where applicable, each FEMA 178 building type would be separated with respect to its diaphragm system. In addition, the poor behavior of multistory multi-unit wood frame buildings with open fronts in both the 1989 Loma Prieta and 1994 Northridge earthquakes led to the definition of the new W1A Model Building Type. A more complete description of the Model Buildings is given in the companion *Example Applications* volume (ATC, 1997).

Significant damage to certain classes of structures occurred in the 1994 Northridge earthquake. Some of the deficiencies that led to the severe damage and, in some cases, collapse of these structures are not completely identified in the FEMA 178 (BSSC, 1992a) list of potential deficiencies. These have been defined and added to the scope of deficiencies addressed by the Simplified Rehabilitation Method, and are suggested as an amendment to FEMA 178. FEMA 178 (BSSC, 1992a) is currently under revision to include both updated information and the Damage Control Structural Performance Range, as well as Life Safety.

The potential for near-field effects—intense shaking and large, damaging velocity pulses in the earthquake source region—has been a topic of discussion for many decades, but only recently were instances specifically observed and recorded. Recent earthquakes in both California and Japan have provided strong motion records that indicate the need to consider stronger ground motions in the near-field area of large earthquakes. It was also observed in these earthquakes that only mid-rise buildings located very close to the source of the earthquake were affected. As a result, the current trend in seismic design guidelines is to include a near-field factor to essentially increase the design lateral force for mid-rise buildings—those with periods greater that 1.0 second—located within ten kilometers of large active faults. Because the Simplified Rehabilitation Method cannot be used on the classes of buildings affected by these near-fault provisions, they need not be considered. They have been properly considered and included in the appropriate sections of the *Guidelines* as they relate to the Systematic Rehabilitation Method.

The lateral force provisions and analysis procedures used in FEMA 178 (BSSC, 1992a) are based on the 1988 *NEHRP Recommended Provisions* for new buildings (BSSC, 1988). As such, they represent the traditional equivalent lateral force procedure that has been used for decades in most seismic codes and guidelines. This procedure is based on the assumption that buildings designed to resist highly reduced (hence called "equivalent") lateral forces within their elastic limits, and properly detailed for ductility, will behave in an appropriate, life-safe manner when subjected to actual earthquake motions. Based on the historic performance of buildings and the levels of ground motion recorded for various earthquakes, current codes reduce the actual forces by *R* factors and include maximum values for purposes of design. These *R* factors (structural response modification factors) and maximum values are based on the judgment and experience of the code and guideline writers (ATC, 1995). When estimates are needed of the forces or deflections caused by the actual earthquake motion, the procedures use a C_d factor to adjust the values up to an appropriate level.

Because of the unique conditions present in existing buildings, the Systematic Rehabilitation Method of the *Guidelines* takes an entirely different approach to the determination of lateral forces and resulting expected deflections. Since most existing buildings needing rehabilitation do not contain the details of construction needed to validate the large reduction factors, a procedure has been developed that allows the individual evaluation of the various building components' capacity to resist the inelastic deformation and strength demands that are expected. In essence, the *Guidelines* define earthquake motions in terms of the expected maximum displacement based on an acceleration response spectrum, and define the pseudo lateral loads needed to cause those expected displacements for use in the evaluation process. Thus, in the Linear Static Procedure of the *Guidelines*, pseudo lateral loads are much larger than those specified traditionally, since they do not include any reduction factors. The appropriate reduction is considered on a component-by-component basis in terms of the allowable capacities, and the *m* value. The reader is referred to Chapter 3 of both the *Guidelines* and the *Commentary* for a complete discussion of the new procedure.

The example given in Chapter 10 of the *Guidelines* (see Figure 10-1) illustrates this point in terms of two hypothetical reinforced concrete structures with perimeter shear walls. The buildings are 120 feet square, and have nine- inch-thick concrete flat slabs at the floors and roof, and eight-inch-thick exterior walls with approximately 30% openings for windows and doors. They are located on S2 soil (FEMA 178 method) or Class C soil (*Guidelines* method). The six-story building is located in an area of low seismicity and the three-story building is located in a High Seismic Zone. The point of proper comparison in the two methods is the ratio of demand/capacity. While the base shears vary by approximately six times, and the allowable capacities by three times, the values of demand/capacity are similar.

As a matter of comparison, the FEMA 178 (BSSC, 1992a) deflections and shears are also plotted with and without their reduction factors. It could be expected that there would be a rough correlation between the unreduced FEMA 178 values and the *Guidelines* values. However, there is a significant difference in the results, primarily due to the basic definition of the pseudo lateral load, the method used to calculate the building period, and the global reduction (0.85 and 0.67) taken in FEMA 178 spectral ordinates to account for the difference in a mean value response spectrum and a mean plus one standard deviation spectrum. This reduction is not taken in the *Guidelines* procedure.

Traditionally, the spectra used to develop the equivalent lateral forces used in codes and guidelines for new buildings are based on a probable earthquake, defined as one with a probability of exceedance equal to 10% in 50 years (10%/50 year), and a related response spectra that represents the mean plus one standard deviation values. In 1987, the Applied Technology Council's ATC-14 report, *Evaluating the Seismic Resistance of Existing Buildings* (ATC, 1987), which served as the basis for FEMA 178, recommended that the spectra for evaluating existing buildings be modified to represent mean values. They argued that, when evaluating existing buildings where there is a high degree of uncertainty and the cost of strengthening is very high, it is more appropriate to use the values associated with the mean probable earthquake than the probable earthquake. This remains a controversial recommendation.

Integrating FEMA 178 evaluation criteria into a rehabilitation guideline has the advantage of separating building elements and systems into individual units, which can be identified relatively quickly and mitigated somewhat independently. This technique works well for simple, low-rise buildings of uniform construction that match the model buildings. For buildings that exhibit complex interaction between elements, such that mitigating one deficiency may only change the weak link in the overall system—or even make another element worse—a more systematic analysis is necessary, and the Simplified Rehabilitation Method is not appropriate.

Certain building systems are excluded from the Simplified Rehabilitation Methods because of their complexity and the possibly unique behavior among individual buildings. Excluded from Simplified Rehabilitation are tall and irregular structures whose behavior is difficult to predict within the provisions of FEMA 178. Buildings that are of hybrid construction (not one of the common building types), including structures with different structural systems in each direction, are also excluded. In addition, the behavior of concrete frame structures, especially in older buildings and in parking garages, has been shown in recent earthquakes to be highly variable, so these buildings are also excluded except in regions of low seismicity. Buildings with significant plan or vertical irregularities have very different characteristics than those expected in regular buildings. Typical analysis methods may not be appropriate in these cases.

The special procedures for evaluating unreinforced masonry buildings presented in Appendix C of FEMA 178 (BSSC, 1992a) allow their use for buildings up to six stories in height. This is consistent with other guidelines, such as the *Uniform Code for Building Conservation* (*UCBC*) (ICBO, 1994a). This may be somewhat nonconservative in higher seismic zones. The *UCBC* is not regarded widely as a document whose goal is life safety, but rather as a hazard reduction guideline. For this reason, the limitations on height for unreinforced masonry (URM) buildings has been reduced in all regions. Height limitations for other building types were developed from comparisons to the values for URMs and to the typical limitations provided by actual construction practice. These limitations apply only when the rehabilitation goal is to achieve the Life Safety Performance Level.

While an engineer may choose to mitigate all of a building's identified FEMA 178 deficiencies by using Simplified Rehabilitation, such a technique should not be considered sufficient to achieve the Basic Safety Objective (BSO) or any Enhanced Safety Objective as defined in Chapter 2. Since the method is based on FEMA 178, which evaluates a building only for compliance with life safety criteria based on a level of earthquake shaking estimated to have a 10% probability of being reached or exceeded in a 50-year period of time (10%/50 year), there is no assurance that it will satisfy the Collapse Prevention criteria as described in this document for the BSO, especially in zones of low seismicity.

The BSO defined in Chapter 2 requires meeting both the Life Safety Performance Level for the BSE-1 (typically, the 10%/50 year) level of motion, and the Collapse Prevention Performance Level for the BSE-2 (typically, the 2%/50 year) level of motion. In regions of low to moderate seismicity, the BSE-2 event may be substantially larger than the BSE-1 earthquake. The attainment of the BSO requires the use of the Systematic Rehabilitation Method, described in Chapter 3, to verify performance for the BSE-2. It is highly recommended that consideration be given to the performance of the rehabilitated building under the BSE-2. Such a consideration need not include a complex, nonlinear analysis, nor should it be based on simple increase in the lateral forces used for design. Rather, it requires that the design professional consider the post-elastic behavior of the building, determine its yielding mechanisms and maximum expected displacements, and determine whether the structure will be subject to collapse when the building is subjected to the BSE-2.

The use of the Systematic Rehabilitation Method is also encouraged if the added cost of a more complex analysis can be offset by a substantial reduction in the cost of the mitigation required.

C10.2 Procedural Steps

The FEMA 178 (BSSC, 1992a) evaluation is intended to stand apart from the Systematic Rehabilitation Method described in the *Guidelines*. Existing elements, systems, and mitigation schemes do not have to be checked using the force levels, *m* factors, analysis techniques, and the like, contained in the Systematic Rehabilitation Method.

FEMA 178 lists specific deficiencies both by Model Building Type and as associated with each building system. *Guidelines* Tables 10-3 through 10-21 further group them by general characteristics. For example, the deficiency listing: "Diaphragm Stiffness/Strength," includes deficiencies related to the type of sheathing used, the diaphragm span, and lack of blocking. Each deficiency group is named and defined in this *Commentary* Section C10.5 and related to all of the FEMA 178 (BSSC, 1992a) deficiencies as amended. *Guidelines* Table 10-22 provides a complete cross-reference.

In addition, within the table for each Model Building Type, each deficiency group is ranked from most critical at the top to least critical at the bottom. For example, in Table 10-14, in a precast/tilt-up concrete shear wall with flexible diaphragm (PC1) building, the lack of positive gravity frame connections (e.g., of girders to posts by sheet metal hardware or bolts) has a greater potential to lower the building's performance (a partial collapse of the roof structure supported by the beam), than a deficiency in lateral forces on foundations (e.g., poor reinforcing in the footings).

The ranking was based on the following characteristics of each deficiency group:

1. Most critical
   a. Building systems: those with a discontinuous load path and little redundancy
   b. Building elements: those with low strength and low ductility
2. Intermediate
   a. Building systems: those with a discontinuous load path but with substantial redundancy
   b. Building elements: those with substantial strength but low ductility
3. Least critical
   a. Building systems: those with a substantial load path but little redundancy
   b. Building elements: those with low strength but substantial ductility

The intention of Tables 10-3 to 10-21 is to guide the design professional so that partial rehabilitation efforts will be useful. For example, if the foundation is strengthened in a PC1 building but a poor girder/wall connection is left alone, relatively little has been done to improve the expected performance of the building. Considerable professional judgment must be used when evaluating a structure's unique behavior and determining which deficiencies should be strengthened and in what order.

Use of the Systematic Rehabilitation Method is encouraged where the FEMA 178 procedures may be unduly conservative. A thorough, wide-ranging solution is often very cost-effective, making up for the extra time spent in the design process.

C10.3 Suggested Corrective Measures for Deficiencies

The application of the Simplified Rehabilitation Method is essentially the performance of a complete FEMA 178 (BSSC, 1992a) evaluation of a building, correcting any deficiencies that are identified. Although FEMA 178 contains "checklists" of potential deficiencies related to a Model Building Type, it is not intended to be used selectively, but rather applied to a building's entire lateral-force-resisting system. It outlines and describes the procedures to follow to perform a thorough analysis and identification of deficiencies.

This section is organized around the major lateral-force-resisting systems common to the Model Building Types, including the overall building configuration, the different vertical lateral-force-resisting systems, diaphragms, connections, and geological considerations. A section is devoted specifically to the evaluation of URM buildings, corresponding to Appendix C of FEMA 178.

Each of the subsections in this section groups the deficiencies identified in FEMA 178 (BSSC, 1992a) into general categories where appropriate, and provides references to specific FEMA 178 sections relating to each deficiency in the group. An expanded discussion of each group is included, with suggestions for additional evaluation techniques beyond those described in FEMA 178, including those found in Systematic Rehabilitation. Suggestions and references for typical rehabilitation strategies are also provided. Table 10-22 cross-references the FEMA 178 (BSSC, 1992a) and *Guidelines* numbers. Section C10.5 of this *Commentary* provides a complete list of the FEMA 178 deficiency evaluation statements, as well as the eight new potential deficiencies presented in the *Guidelines*, Section 10.4.

C10.3.1 Building Systems

C10.3.1.1 Load Path

A complete load path for the transmission of forces from the point where they are generated to the foundation and supporting soil material is essential for the proper seismic behavior of a structure. If there is a discontinuity in the load path, the building is unable to resist earthquake-induced forces, regardless of the strength of existing elements. (FEMA 178 [BSSC, 1992a], Section 3.1.)

The first step in finding missing links in a load path is to identify the location of loads generated throughout the building. These loads generate forces and moments. The loads are traced through the structure, usually beginning with the diaphragms, proceeding to the vertical lateral-force-resisting systems (walls or frames) through connections, and into the foundation through connections. Certain loads are local, such as bending moments generated in a diaphragm, and are not transferred to the foundation. In cases where there is a structural discontinuity, a load path may exist but it may be a very undesirable one, such as with offset shear walls, which transfer overturning moments through beam or frame elements not intended to be part of the lateral-force-resisting system. Identification of undesirable load paths in a complex structure can be facilitated with the development of appropriate computer modeling.

If the existing load path is complete but potentially undesirable, it may be possible to show that, while not ideal, the existing load path is acceptable. It may also be possible, using the Systematic Rehabilitation Method described in Chapters 2 and 3, to show that alternate load paths can be developed if the primary path is discontinuous or insufficient.

C10.3.1.2 Redundancy

To account for uncertainties in both the expected loads and the analysis methods—and in the inability to know precisely the existing condition of all structural elements—it is essential that buildings contain redundancy in their lateral-force-resisting systems. Redundancy ensures that if a single element—such as a brace, moment connection, or shear wall pier (or entire wall line if it is small)—fails for any reason, the structure has alternative paths by which lateral forces can be resisted. (FEMA 178 [BSSC, 1992a], Section 3.2.)

It is not sufficient to show by analysis that under the design forces (or even a multiple of the design forces) no structural elements yield, because the unknowns associated with the building and the ground motion are potentially large and the consequences of failure significant. Analysis for redundancy should show that if major elements are seriously damaged, a complete load path remains. In this analysis, the engineer does not have to show that the remaining elements are sufficient to resist the design lateral loads. The Nonlinear Static Procedure (Chapter 3) can be used to investigate whether the failure of a single element causes an instability.

C10.3.1.3 Vertical Irregularities

Vertical irregularities in a building may result in a concentration of forces or deflections or in an undesirable load path in the vertical lateral-force-resisting system. In extreme cases, this can result in serious damage to or collapse of a building, since the lateral system is often integral with the gravity-load-resisting system. Vertical irregularities typically occur in a story that is significantly more flexible or weaker than adjacent stories. The irregularity can also occur where there is a significant change in building dimension over its height, such as with setbacks, where there are large concentrations of mass, or where vertical elements are discontinuous in a story.

The use of simplified procedures for determining the significance of vertical irregularities is difficult, especially in tall or complex buildings. The deficiency may be difficult to spot in a visual survey or with simple calculations. The Quick Check procedures in FEMA 178 for calculating story capacities, forces, and drifts can be used to determine the presence of a vertical irregularity, but should be verified through a complete analysis.

While it is possible in some cases to allow the irregularity to remain and to strengthen those structural elements that are insufficient, this may require substantial additional analysis, and does not address the problem directly nor in a manner that is permitted by the Simplified Rehabilitation Method. Because the presence of a vertical irregularity in a single story can affect the force and deflection characteristics of the entire building, dynamic or nonlinear analysis techniques are usually required to evaluate the consequences.

By using one of the procedures in the Systematic Rehabilitation Method, the presence of a vertical irregularity often can be determined to be inconsequential. (FEMA 178 [BSSC, 1992a], Sections 3.3.1 through 3.3.5.)

C10.3.1.4 Plan Irregularities

Horizontal irregularities in the structural system of a building typically result in torsion caused by a differential between the center of mass and the center of rigidity in a story, and may result in undesirable dynamic behavior, including building rotation or excessive deflection at the more flexible building ends. Such plan irregularities, hereafter called “torsional irregularities,” can lead to excessive and concentrated demands on the diaphragms that are often not of adequate strength and are not otherwise identified in the Simplified Rehabilitation Method.

It is often possible to determine the presence of torsional irregularities using simplified procedures such as a relative rigidity analysis. As torsion is the primary horizontal irregularity, the deficiency may not be difficult to spot in a visual survey or with simple calculations. Where adjacent stories affect the stiffness properties of the story in question, or where the irregularity is caused by re-entrant corners, systematic analysis may be warranted.

Using a nonlinear procedure in Systematic Rehabilitation, the presence of a torsional irregularity often can be identified and possibly determined to be insignificant. If the irregularity cannot be eliminated, it may be possible, using these methods, to identify the elements that need to be strengthened as a result of the irregularity.

Other plan irregularities related to the plan configuration of the building require consideration of the interconnection of the building at the re-entrant corners, the strength of diaphragms, and the overall lateral system for each wing. Each of these is addressed later by other potential deficiencies. (FEMA 178 [BSSC, 1992a], Section 3.3.6.)

C10.3.1.5 Adjacent Buildings

Adjacent structures can pound in an earthquake if they are too close and they exhibit different dynamic deflection characteristics. The structures may be part of a single complex of buildings or two buildings separated by a property line. Pounding damage can be especially severe if the floors of adjacent buildings do not line up or one building is significantly taller than the other. In these instances, the floor of one building, which is typically very stiff, pounds into the wall of the other, which is usually very flexible out-of-plane. In severe cases, pounding has led to collapse or partial collapse of one of the two buildings.

The Quick Checks for drift in FEMA 178 (BSSC, 1992a) are used to identify the possibility of pounding, since the actual drifts in a building are much higher than those computed directly from the forces used for designs. Design forces based on reduced accelerations from the elastic earthquake spectrum anticipate some yielding in the elements and therefore will lead to larger expected drifts. Expected drift can be more accurately calculated when based on the actual expected earthquake accelerations using advanced techniques. Chapter 3 provides Analysis Procedures for obtaining more realistic estimates of drift.

The Nonlinear Dynamic Procedure described for use with Systematic Rehabilitation may be used in complex or tall buildings to make a more accurate determination of story drift capacity versus demand. (FEMA 178 [BSSC, 1992a], Section 3.4.)

C10.3.1.6 Lateral Load Path at Pile Caps

This is an amendment to the FEMA 178 (BSSC, 1992a) deficiency lists. Refer to Section 10.4.1.1 of the *Guidelines* for the evaluation statement, comment, and procedure.

C10.3.1.7 Deflection Compatibility

This is an amendment to the FEMA 178 (BSSC, 1992a) deficiency lists. Refer to Section 10.4.1.2 of the *Guidelines* for the evaluation statement, comment, and procedure.

C10.3.2 Moment Frames

C10.3.2.1 Steel Moment Frames

A. Drift

Moment-resisting frames are generally more flexible than shear wall or braced frame structures, and are likely to sustain larger lateral building displacements (total and inter-story drifts). Large inter-story drifts in structures can generally be expected to cause more extensive nonstructural damage to elements such as partitions and cladding; potentially significant P-Δ effects in taller structures; damage to welded beam-column connections; and pounding where there are closely adjacent buildings.

The Quick Check for drift in FEMA 178 (BSSC, 1992a) can be used for short, simple buildings to identify structures that may be susceptible to excessive inter-story drifts. The drifts calculated from the Quick Check will be much smaller than the actual drifts caused by the earthquake, since the calculations are based on the basic equivalent lateral force procedure rather than the expected accelerations and displacements of the earthquake ground motion. The allowable drift values in FEMA 178 (BSSC, 1992a) are intended to take this into account. All buildings failing the Quick Check should be fully analyzed, using the Systematic Rehabilitation Method.

The Systematic Rehabilitation Method should be used in tall and/or irregular buildings to make a more accurate determination of inter-story drifts. (FEMA 178 [BSSC, 1992a], Section 4.2.1.)

B. Frames

Proper performance of steel moment-resisting frames depends on the ability of all of the various elements of the lateral-force-resisting system to develop required member strengths and meet local ductility demands. Without this ability, the frames will be subject to unacceptable damage. As such, the frame elements need to be rehabilitated in a way that will meet both strength and deformation demands.

Structural steel sections are proportioned to maximize their efficiency. This makes them more susceptible to stability concerns (both local and global) than other structural materials. Use of compact sections, which have proper width-to-thickness ratios for the various portions of the cross sections, will delay the onset of local buckling and permit proper inelastic response. Global stability concerns need to be met by providing adequate lateral bracing at locations of expected plastic hinging and other code-specified intervals. Large local member discontinuities, such as web penetrations in beams, may also reduce member strength and deformation capacities.

Evaluation of the impact of noncompact members and members with large web penetrations can be made using procedures provided by the AISC specifications (AISC, 1986, 1989). Stability analyses may be required to determine the effects of lateral bracing that is less than the typically specified requirements. Since inelastic deformation capacities are not explicitly addressed in the calculation procedures recommended by FEMA 178 (BSSC, 1992a), the calculation of member force demands should be done using elevated lateral force levels (by using reduced R factors—structural response modification factors) to account partially for the reduced deformation capacities. Evaluation using Systematic Rehabilitation will therefore likely be required, in order to estimate the effects of these considerations on the member deformations that can actually be accommodated unless the deficient members are rehabilitated.

Systematic Rehabilitation should be used in buildings with significant stability concerns to obtain realistic estimates of the member demands. Detailed evaluation of the element deformation capacities and stability will also be required. (FEMA 178 [BSSC, 1992a], Sections 4.2.2, 4.2.3 and 4.2.9.)

C. Strong Column-Weak Beam

One goal for well-configured moment frame systems is to distribute inelastic action throughout the lateral-force-resisting elements, which requires the capacity of the column at any moment frame joint be greater than the capacity of the beams. In conditions where the beams are stronger than the columns, column hinging can lead to story mechanisms, which can result in an excessively large drift within a single story. The large inelastic rotation demands that result could jeopardize the stability of the frame, due to P-Δ effects. Column hinging is also considered undesirable, since large gravity loads may be supported by a column. A beam, on the other hand, supports a significantly smaller portion of the gravity loads on the structure. Local hinging in the beams will therefore affect a much smaller portion of the building. (FEMA 178 [BSSC, 1992a], Section 4.2.8.)

FEMA 178 prescribes that local joint analyses be performed to evaluate these effects. The effects of gravity forces on the member capacities must be considered. Axial force effects on the columns—due to both gravity forces and frame overturning effects—will reduce the residual capacity for resisting seismically-induced bending moments. The supplemental beam strength provided by the composite action between the concrete floor slabs and the steel beams has generally not been considered, but may be significant in some instances.

The Systematic Rehabilitation Method, including nonlinear procedures and dynamic procedures, should be used in tall and/or irregular buildings to determine whether the potential for the development of story mechanisms exists. Proper consideration of slab effects and column overturning effects is also necessary.

D. Connections

Prior to the 1994 Northridge earthquake, steel moment frame connections consisting of full penetration flange welds and a bolted shear tab were thought to be ductile and capable of developing the full capacity of the beam section. This connection detail, which became almost an industry standard in the period from 1970 to 1995, experienced serious damage in the form of weld and beam or column fractures in over 100 buildings as a result of the Northridge earthquake. Because of this, an emergency code change was made to the 1994 *UBC* (ICBO, 1994b), which removed the "prequalification" of this connection detail. The newly discovered susceptibility of this detail is the focus of a great deal of effort to understand the causes of the damage and to develop methods to design, evaluate, and rehabilitate these structures. Previous laboratory testing on partial penetration column splices has shown little or no ductility. No damage to column splices was noted in the Northridge earthquake, although the 1995 Hyogoken-Nanbu (Kobe), Japan earthquake did produce a number of such failures. Panel zone doubler plates and continuity plates were also damaged in the Northridge earthquake, although to date their design has not been seen as a significant factor.

Because of the Northridge earthquake damage, the use of FEMA 178 procedures related to welded steel moment frame connections needs to be completely revised. The Systematic Rehabilitation Method provided in these *Guidelines* should be followed. Testing of mock-up connection subassemblages may need to be considered for conditions where no previous test results adequately model the conditions being evaluated. The SAC Joint Venture (whose participants are the Structural Engineers Association of California, the Applied Technology Council, and California Universities for Research in Earthquake Engineering) Program to Reduce the Earthquake Hazards of Steel Moment Frame Structures, and other efforts, are specifically addressing this problem. The most recent publications now recommend that girder flange continuity plates be provided in all cases to reduce the stress concentrations that occur at the web location in the flange welds. See SAC (1995).

At the time of this writing, appropriate systematic solutions are under development by the SAC Steel Program. Such solutions will likely evolve from advanced methods of analysis—such as nonlinear time-history analysis, both on the frame elements and, possibly, on individual joint subassemblages—as well as from extensive additional testing. Because of the variability of construction quality encountered in the post-Northridge inspections, it is likely that the procedure will be explicitly probability-based. At the time of this writing, the latest available guidance is the *SAC Interim Guidelines* (SAC, 1995). (FEMA 178 [BSSC, 1992a], Sections 4.2.4 through 4.2.7.)

C10.3.2.2 Concrete Moment Frames

A. Frame and Nonductile Detail Concerns

Quick Checks. The Quick Checks of FEMA 178 provide generally conservative estimations of shear and drift in the frames, providing the engineer with a "ballpark" estimate of the situation. They are best applied to regular multistory buildings.

Where the initial Quick Check indicates average column shear stress above 60 psi, or if the building is not regular, FEMA 178 refers to the need for a more detailed evaluation. For structures satisfying the limits of Table 10-1, the more detailed evaluation may utilize FEMA 178 forces and procedures. (FEMA 178 [BSSC, 1992a], Sections 4.3.1, 4.3.2.)

Frames. These concerns focus on those elements whose local failure can lead directly to collapse or partial collapse of the building, i.e., precast frames, frames with eccentric joints, and shear-critical columns (shear failure occurs before flexural failure).

In general, prestressed frames should not be justified using Simplified Rehabilitation. It may be possible to show that eccentric joints and shear-critical columns are acceptable by demonstrating that the available shear capacity exceeds the anticipated demand by a significant margin—a factor of approximately 3.0. Reliance on Simplified Rehabilitation to address these concerns should be done with caution and should take into account the structural response as a whole. (FEMA 178 [BSSC, 1992a], Sections 4.3.3, 4.3.4, 4.3.5.)

Strong Column-Weak Beam. Where the sum of the moment capacities of the beams exceeds that of the columns, the failure is likely to occur in the column. This condition is even more critical when the column is shear-critical (see above), because the shear imposed on the column is governed not by the column's flexural capacity but by the capacity of the beams.

Nonductile Detail Concerns. Nonductile frames are elements that do not incorporate the following items addressed in current ductile detailing provisions:

- Anchorage of beam stirrups and column ties into the concrete core with 135-degree hooks
- Close spacing of column ties
- Length and confinement of column bar splices
- Continuity of top and bottom beam bars through the column-beam joint
- Length and location of beam bar splices; close spacing of beam stirrups
- No reliance on bent longitudinal bars for shear reinforcement
- Use of column ties in exterior column/beam joints
- No flat slab/plates working as a beam in frame action

Ductile detailing allows the elements to maintain vertical-load-carrying capacity as the frame displaces beyond the elastic limits of the system and forms plastic hinges.

Current ductile detailing practices have evolved only since the mid-1970s. In general, most frame buildings built before 1973 will likely have nonductile detailing. In some cases, columns were spiral reinforced, which usually provides significant ductility in the columns. However, column bar splices, beam reinforcement, and beam-column joints still need to be evaluated.

Where nonductile components remain essential links in the load path, Systematic Rehabilitation must be used. Careful consideration must be given to the brittle nature of the columns and joints. (FEMA 178 [BSSC, 1992a], Sections 4.3.7 through 4.3.15.)

B. Precast Moment Frames

Precast concrete frames without shear walls may not be addressed under Simplified Rehabilitation (see Table 10-1). Where shear walls are present, the precast connections often govern the performance and need to be carefully evaluated. If the connections are configured such that yielding occurs within the members rather than in the connections, the building should be evaluated as a shear wall system. (FEMA 178 [BSSC, 1992a], Section 4.4.1.)

C10.3.2.3 Frames Not Part of the Lateral-Force-Resisting System

A. Complete Frames

Typically, incomplete frames are essentially bearing wall systems. Damage to the wall may lead to a loss of gravity load resistance. The evaluation should utilize FEMA 178 (BSSC, 1992a) forces and procedures and should include a check of the connection between drag elements (i.e., horizontal reinforcement) and the bearing walls.

Strengthening the wall to reduce the stress under combined gravity and seismic loads may be more appropriate when there is nearly enough existing vertical-load-resisting strength. The addition of columns to complete the gravity load path is the preferred solution because it separates the lateral-force-resisting system and damage it may suffer from the vertical-load-resisting system. Where the wall cannot be strengthened nor columns added, the Systematic Rehabilitation Method should be used, since walls and adjacent columns will probably not have ductile detailing. (FEMA 178 [BSSC, 1992a], Section 4.5.1.)

B. Short Captive Columns

See the *Guidelines* Section 10.4.2.2 for explanation of this addition to the FEMA 178 (BSSC, 1992a) potential deficiency list.

C10.3.3 Shear Walls

C10.3.3.1 Cast-in-Place Concrete Shear Walls

A. Shearing Stress

The shearing stress check provides a quick assessment of the overall level of shearing stress in the building’s walls.

Where the average stress exceeds the FEMA 178 (BSSC, 1992a) recommended values, a more detailed evaluation is needed. This detailed evaluation, utilizing FEMA 178 forces and procedures, should account for vertical and horizontal distribution of the seismic forces. Allowable stresses compatible with ACI provisions (ACI, 1989) should be used.

Where the shearing stress limit calculated with the more detailed evaluation is still exceeded, the appropriate

Simplified Rehabilitation solution is to add sufficient shear walls to satisfy the stress check or detailed evaluation criteria. These calculations tend to be very conservative and an appropriate Systematic Rehabilitation should be used if extensive rehabilitation appears to be needed.

Appropriate Systematic Rehabilitation solutions will also address the impact of boundary element configuration on the shear capacity of the walls. (FEMA 178 [BSSC, 1992a], Section 5.1.1.)

B. Overturning

Tall, slender shear walls may have limited overturning resistance. Displacements at the top of the building will be greater than those anticipated by simplifying equations and/or analytical models, if the overturning forces are not properly resisted. Often, sufficient resistance is available in the immediately adjacent columns.

If an extensive amount of work is needed, procedures of the Systematic Rehabilitation Method should be used that include developing analytical models that reflect the load/displacement curves for slender walls, and their interaction throughout the building. (FEMA 178 [BSSC, 1992a], Section 5.1.2.)

C. Coupling Beams

Coupling beams act to tie or couple adjacent walls acting in the same plane. When properly detailed and proportioned, coupling beams have a significant effect on the overall stiffness of the coupled walls and their resistance to overturning.

Appropriate evaluation techniques include first evaluating the walls acting without coupling. This evaluation includes shears, moments, and wall stability. If the walls are stable and satisfy Simplified Rehabilitation wall criteria, the approach would then focus on preventing debris from becoming a falling hazard. (FEMA 178 [BSSC, 1992a], Section 5.1.3.)

D. Boundary Component Detailing

Fully effective shear walls require the following boundary element components to be appropriately detailed: (1) steel column splices, (2) steel column/ concrete wall shear transfer mechanism, and (3) confinement ties at vertical reinforcement. Brittle failure of any one of these components can lead to substantially lower wall capacity.

In the Simplified Rehabilitation evaluation, column splices, shear transfer mechanisms, and confinement should be adequate to develop the amplified FEMA 178 forces.

In Systematic Rehabilitation, reduced capacity of the components can be accounted for. (FEMA 178 [BSSC, 1992a], Sections 5.1.4 - 5.1.6.)

E. Wall Reinforcement

The reinforcement in shear walls controls the ability of the wall to behave appropriately under seismic loads. Openings may significantly interrupt the flow of stresses so that special steel is required around the boundaries.

In the Simplified Rehabilitation evaluation, use forces and procedures outlined in FEMA 178 (BSSC, 1992a).

In Systematic Rehabilitation, the shear walls can be modeled to reflect the anticipated degradation of the wall and, in some cases, allow isolated walls without enough strength to remain without strengthening because there is available strength elsewhere, in other walls. (FEMA 178 [BSSC, 1992a], Sections 5.1.7, 5.1.8.)

C10.3.3.2 Precast Concrete Shear Walls

A. Panel-to-Panel Connections

Welded steel inserts can be brittle and may not be able to transfer the overturning forces between panels. Latent stresses may be present due to shrinkage and temperature effects.

The Simplified Rehabilitation evaluation should follow the procedures outlined in FEMA 178 (BSSC, 1992a). Particular care must be taken to ensure that there is substantial strength available in the as-built connections to resist the actual earthquake forces, since these connections typically have no ductility. It is preferable for the connections to be able to develop the full yield strength of the panel.

B. Wall Openings

In tilt-up construction, walls with large openings require special detailing for collector elements, shear transfer, and overturning. Often, the piers and spandrels were detailed only as walls and not as elements of a lateral-force-resisting concrete frame.

Panel connections should be assessed. If the panel connections are strong enough, the panels will behave like a moment frame, and each element should be evaluated for frame action. It is unlikely that panels with large openings can be shown to be adequate when considered as moment frames.

C. Collectors

Where collectors are needed to transfer lateral forces out of the diaphragm into the shear walls, the collector and its connections should be evaluated using FEMA 178 (BSSC, 1992a), Section 4.4.2, to determine whether they are adequate to develop the design forces. Full consideration should be given to existing continuous slab and beam reinforcing that may naturally serve the collector purpose.

C10.3.3.3 Masonry Shear Walls

A. Reinforcing in Masonry Walls

If there is any possible evidence of reinforcing in masonry walls, or if the standard construction techniques for the region include reinforcing masonry construction, then every effort should be made to identify and take full account of the level of reinforcing. This is especially true for concrete block construction.

Consideration of the building's adequacy as a URM building should precede the addition of new reinforced masonry or shotcrete walls.

B. Shearing Stress

A detailed analysis of the lateral-force-resisting walls should be performed, using the provisions of *NEHRP Recommended Provisions for Seismic Regulations for New Buildings* (BSSC, 1995a). The allowable stresses as specified in MSJC (1995) should be used, multiplied by 2.5 times a capacity reduction factor. (FEMA 178 [BSSC, 1992a], Section 5.3.1.)

In order to utilize MSJC (1995), the prism strength of the masonry and the yield strength of the steel must be established. If the mortar type is lime-sand-mortar or a lime-sand-portland cement mortar, and the approximate strength of the masonry unit can be established, then a reasonable lower bound value, using the tables in MSJC (1995), can be assumed for the prism strength. The yield strength of the reinforcing can be conservatively estimated as 30,000 psi.

C. Reinforcing at Openings

Masonry control joints are sometimes located at openings. The presence of a control joint, large shrinkage cracks, or a steel or precast concrete lintel would indicate that trim reinforcing was not installed.

D. Unreinforced Masonry Shear Walls

The evaluation of URM buildings is based upon the Simplified Rehabilitation Method and consists of using the provisions of FEMA 178 (BSSC, 1992a).

The evaluation is based upon a reduced base shear, building evaluation checklists, and a series of Quick Checks to determine if the strength of the building is satisfactory. In the event that the structure does not pass the Quick Check procedure, it is recommended that the engineer use the Systematic Rehabilitation Method outlined in the *Guidelines*.

An evaluation can also be made using Appendix C of FEMA 178 (BSSC, 1992a). However, the performance objective of Appendix C is significant hazard reduction, which is a lower objective than assumed for the Life Safety Performance Level. In order to comply with the quality control requirements of Appendix C, testing of masonry and anchors is required.

The composition of the wall must be determined in order to compute the shearing stresses in the wall and the thickness that is to be used to resist out-of-plane forces. The lay-up of the walls is deficient if significant voids are left between the wythes. In this case, the walls may not be able to resist out-of-plane forces as expected, due to a lack of composite action between the inner and outer wythes. Appendix C is based upon brick construction. Consequently, there is no procedure established to test concrete masonry units. Appropriate testing is needed. When the net area is required for shearing stress computations, a section of the wall should be removed in order to establish the bedding area. Walls with insufficient thickness should be either strengthened by increasing the thickness, or removed. (FEMA 178 [BSSC, 1992a], Sections 5.4.1, 5.4.2.)

E. Proportions of Solid Walls

The out-of-plane requirements for infill walls also apply to unreinforced masonry bearing walls.

Height-to-thickness ratios are established for areas with ground acceleration greater than 0.2g in Section 5.5 of

FEMA 178 (BSSC, 1992a); areas of acceleration less than 0.2g are covered in Appendix C of FEMA 178.

The procedure to check walls that do not meet the height-to-thickness ratios (Section 2.4.6) for out-of-plane forces in areas with a design acceleration less than 0.2g requires the evaluation of the seismic demand on the wall and calculations to determine the bending stresses.

The MSJC (1995) provisions allow flexural tension in the wall when the building is in moderate seismic areas and the wall is unreinforced or has minimal prescriptive reinforcement. If the construction does not conform to the MSJC (1995) minimum reinforcing requirements, the MSJC allowable stress, multiplied by 2.5, and reduced by the appropriate capacity reduction factor, may be used to determine the flexured capacity.

F. Infill Walls

The shear capacity of the reinforced concrete columns constrained by the infill should be determined using the Quick Check procedures of FEMA 178 (BSSC, 1992a). This check neglects the shear resistance provided by the column ties.

If the column fails the Quick Check, the location and size of the reinforcing and the strength of the concrete should be determined. The column should be analyzed for the capacity to resist the imposed moments and shears, using a more detailed evaluation. If the column is adequate as a “short column,” the partial height infill wall can be connected to the columns and considered to span horizontally. Otherwise, isolation is required.

C10.3.3.4 Shear Walls in Wood Frame Buildings

A. Shear Stress

All walls in wood frame construction participate in the lateral-force-resisting system. The evaluation of these walls is based on the FEMA 178 (BSSC, 1992a) Quick Checks. Where the average stress exceeds the FEMA 178 recommended values, a more detailed evaluation is needed. This detailed evaluation, using FEMA 178 (BSSC, 1992a) forces and procedures, should employ a more accurate estimation of the level and distribution of the lateral loads.

B. Openings

When walls have large openings, little or no resistance is available and they must be specially detailed or braced to other parts of the structure. Such bracing is not a conventional construction procedure. Lack of this bracing can lead to collapse of the wall.

It is necessary to check the ability of the walls and diaphragms to control, through torsional capacity, displacements at walls with large openings. A check should also be made to determine that the diaphragm is a complete system with chords and collectors provided to deliver the lateral loads as required.

C. Wall Detailing

The basic lateral strength and stability of wood walls is limited. Additional strength can be achieved if the wall supports enough dead load to resist overturning and has details adequate to transfer these loads.

D. Cripple Walls

Cripple walls are short stud walls that enclose a crawl space between the first floor and the ground. Often there are no other walls at this level, and these walls have no stiffening elements other than decorative sheathing. If this sheathing fails, the relatively rigid upper part of the building will fall. To be effective, all exterior cripple walls below the first floor level should be checked to ensure that they have adequate shear strength and stiffness, and proper connection to the floor and foundation. Cripple walls that change height along their length, such as in hillside locations, do not distribute shear uniformly to the walls, due to the varying stiffness, and create significant torsion in the building foundation. Simply sheathing all surfaces may not provide adequate strength and stiffness. On extreme slopes, rigid bracing using steel braces, reinforced masonry shearwalls, or concrete shearwalls may need to be added.

E. Narrow Wood Shear Walls

See *Guidelines* Section 10.4.3.1.

F. Stucco Shear Walls

See *Guidelines* Section 10.4.3.2.

G. Gypsum Wallboard or Plaster Shear Walls

See *Guidelines* Section 10.4.3.3.

C10.3.4 Steel Braced Frames

C10.3.4.1 System Concerns

Braced frame structures are inherently stiffer than moment frame structures, since they resist lateral forces through truss action.

The Quick Stress Checks in FEMA 178 (BSSC, 1992a) can be used for simple buildings to assess the strength provided by the braced frames. Consideration of gravity effects on beams and columns in these frames should be combined with the lateral forces in a simplified analysis. Note that this check does not provide any indication of the ductility of these frames, which is also necessary for proper seismic performance. This tool is not appropriate for tall and/or irregular buildings.

Systematic Rehabilitation should be used in tall and/or irregular buildings to determine the expected frame capacity versus demand. (FEMA 178 [BSSC, 1992a], Section 6.1.1.)

C10.3.4.2 Stiffness of Diagonals

Code design requirements have allowed compression diagonal braces to have Kl/r ratios of up to 200 (Kl is the effective length; r minimizes the moment of inertia). Tension-only bracing is also allowed for some buildings. Cyclic tests have demonstrated that elements with high Kl/r ratios subjected to large deformations cannot be expected to provide adequate performance. Tension-only systems may allow the brace to deform with large velocities during cyclic response after tension yielding cycles have occurred. Limited energy dissipation and premature fracture can significantly increase the building displacements and jeopardize the performance of the framing system.

Simple braced frame analysis tools are provided by FEMA 178 (BSSC, 1992a), with a 25% amplification of the seismic forces prescribed where bracing elements have a Kl/r ratio greater than 120. This procedure is intended to require braced frames with relatively flexible diagonals to be capable of resisting larger forces. Differences in the performance of elements of various cross sections (e.g., cold-formed tubes, pipes, double angles or channels, single angles), can also be significant to the cyclic deformation performance and should be considered in the analysis.

Systematic Rehabilitation should be used in tall and/or irregular buildings to determine the expected frame capacity versus demand. Estimation of deformation capacities of bracing elements can be made based on examination of past experimental investigation results. (FEMA 178 [BSSC, 1992a], Sections 6.1.2 and 6.1.3.)

C10.3.4.3 Chevron or K-Bracing

There are many possible configurations for the diagonal elements in a braced frame. Some systems— chevron or V-braced—raise a concern that is not present for other brace configurations. When the compression brace buckles, the ability of the adjacent tension brace to resist additional load is dependent on the capacity of the floor beam to resist the large vertical load—the vertical component of the force in the tension brace. In most cases, the beams have not been designed for these large forces. As a result, the lateral load performance of these systems is considered to be less desirable than that of X-braced or single diagonal systems. K-bracing, where the diagonal members meet within the height of a column, is even less desirable than chevron bracing, since compression brace buckling can result in a large lateral force on the column, which could jeopardize its stability.

FEMA 178 (BSSC, 1992a) prescribes higher force levels for K-braced frames in an attempt to reduce the deformation demands to which the column may be subjected. No specific procedures are provided for chevron or V-braced frames.

Systematic Rehabilitation should be used in tall and/or irregular buildings to determine the expected frame capacity versus demand. (FEMA 178 [BSSC, 1992a], Section 6.1.4.)

C10.3.4.4 Braced Frame Connections

It is generally considered advisable to make the connections between the members of seismically designed frames stronger than the members, since connection failure is generally not ductile and may result in separation of the parts. Member yielding is generally considered to be more desirable than inelastic response of the connections. Especially important connections in braced frames are the column splices, since they may be subject to large tensile forces that could jeopardize stability if the connection were to fail. Proper consideration of any eccentricities between the connected members is necessary to avoid yielding prior to the development of the member strength.

FEMA 178 (BSSC, 1992a) requires that the brace connections be capable of developing the capacity of the diagonals, or else an amplified seismic load must be used. Special requirements for column splices are noted, with increased demands specified for partial penetration splices that have not demonstrated significant ductility in laboratory testing. Any eccentricities in the connections of the braced frames must be properly analyzed to ensure that premature member yielding due to the eccentricity does not occur.

Systematic Rehabilitation Analytical Procedures should be used in tall and/or irregular buildings to determine the expected frame demands. (FEMA 178 [BSSC, 1992a], Sections 6.1.5, 6.1.6, and 6.1.7.)

C10.3.5 Diaphragms

C10.3.5.1 Re-entrant Corners

Diaphragms with plan irregularities such as extending wings, plan insets, or E-, T-, X-, and L-shaped configurations have re-entrant corners where large tensile and compressive forces can develop. The diaphragm may not have sufficient strength at these re-entrant corners to resist these tensile and compressive forces, and locally concentrated damage may occur.

The chord requirements at the re-entrant corners of the diaphragm should be calculated from the required shear force that the diaphragm must resist, the configuration of the diaphragm, and the location of the vertical lateral-force-resisting elements (e.g., moment frames, braced frames, shear walls). Any chords and chord connections that may exist must be evaluated to determine if they have sufficient capacity to resist the required tensile and compressive forces at the re-entrant corner.

C10.3.5.2 Crossties

Continuous crossties between diaphragm chords are needed to resist out-of-plane forces on the walls and transfer these forces through the diaphragm into the supporting walls or frames. It is critical that the crossties have a positive and direct connection to the laterally supported walls that will prevent the walls and the diaphragm from separating. The connection of the crosstie to the wall and connections within the crosstie must be designed so cross-grain bending or cross-grain tension is not present in any wood member. Subdiaphragms may be used to reduce the length of some of the crossties, but full crossties must still be provided between subdiaphragms.

The out-of-plane wall anchorage force that the crossties are required to resist should be calculated. Both the crossties and a positive direct connection between the wall and the crossties should be designed to resist the required force without cross-grain bending or cross-grain tension in any wood members.

C10.3.5.3 Diaphragm Openings

Openings in diaphragms cause an increased shear demand in the segments of the diaphragm adjacent to the opening. Tension and compression forces caused by bending moments are at the edges of these segments of the diaphragm. Openings that are small relative to the diaphragm depth will cause only a slight increase in the shear demand. Openings that are large relative to the diaphragm depth can result in excessive shear demand and large moments and forces in the diaphragm. The stiffness of a diaphragm with openings of significant size is less than that of a comparable diaphragm without openings.

The shear capacity of the segments of the diaphragm adjacent to the opening should be checked to see if they have sufficient capacity to resist the required shear force, and, if the opening is adjacent to a vertical lateral-force-resisting element, a check should be made to confirm that there is a complete load path with sufficient strength to deliver the diaphragm shear to it. The moments and forces in the segments of the diaphragm adjacent to the opening, and the adequacy of any chords or drag struts, should also be checked.

C10.3.5.4 Diaphragm Stiffness/Strength

A. Board Sheathing

Straight-sheathed diaphragms are very flexible and have low shear capacity when compared to other types of wood diaphragms. Individual boards in the straight-sheathed diaphragm must have at least two nails into each of the supporting members to develop the nail couple, which provides the limited shear capacity of these diaphragms. Because of the limited strength and stiffness of these diaphragms, they are most suitable in areas of low seismicity. In areas of moderate to high seismicity, the span between vertical elements and the span-to-depth ratio of straight-sheathed diaphragms should be limited or the diaphragm should be strengthened. Other considerations include the type of vertical elements—because wood-frame walls tolerate much greater diaphragm deformations than do masonry walls—and the size of the loads, which may be small for many roof diaphragms even in areas of high seismicity.

The shear force that the diaphragm is required to resist should be calculated, and an analysis made to determine if the diaphragm has sufficient strength and stiffness to resist this force.

B. Unblocked Diaphragms

Wood structural panel diaphragms may or may not have blocking at the panel edges that are perpendicular to the framing and not supported by the framing. The shear capacity of unblocked wood structural panel diaphragms is quite limited, due to the reduced shear transfer capacity between panels at the unblocked panel edges. Unblocked diaphragms are also more flexible than comparable blocked diaphragms and will experience increased lateral deflections.

C. Spans

Diaphragms with long spans between vertical elements will often experience large lateral deflections and excessive diaphragm shears. Large deflection in the diaphragm can result in increased damage or collapse of elements laterally supported by the diaphragm. Excessive diaphragm shears will cause damage and reduced stiffness in the diaphragm.

D. Span-to-Depth Ratio

Diaphragms with a high span-to-depth ratio will experience higher flexibility and diaphragm shear than comparable diaphragms with a low span-to-depth ratio. This is especially true for span-to-depth ratios greater than three to one. Large deflection in the diaphragm can result in increased damage or collapse of elements laterally supported by the diaphragm. Excessive diaphragm shears will cause damage and reduced stiffness in the diaphragm.

E. Diaphragm Continuity

Split level floors and roofs or diaphragms interrupted by expansion joints create discontinuities, unless special details are used or lateral-force-resisting elements are provided at the vertical offset of the diaphragm or on both sides of the expansion joint. Such a discontinuity may cause the diaphragm to function as a cantilever element or three-sided diaphragm. If the diaphragm is not supported on at least three sides by lateral-force-resisting elements, torsional forces in the diaphragm may cause it to become unstable. In both the cantilever and three-sided cases, increased lateral deflection in the discontinuous diaphragm may cause increased damage to, or collapse of, the supported elements.

F. Chord Continuity

Diaphragms with discontinuous chords or without chords will be more flexible and will experience more damage at perimeter areas than diaphragms with chords that are continuous and have sufficient connection capacity. Vertical offsets or elevation changes in a diaphragm often cause a chord discontinuity. This is especially critical in wood diaphragms that lack any natural tensile capacity.

C10.3.6 Connections

C10.3.6.1 Diaphragm/Wall Shear Transfer

The diaphragm shear at each floor or roof must be connected to the shear wall in order to provide a complete load path for the shear to transfer. Where the wall does not extend the full depth of the diaphragm, collectors or drag/strut components are required to deliver the shear to the wall.

After calculating the shear force at the shear wall, this force should be divided by the length of the wall to determine the shear transfer connection required per foot of wall. Where the wall does not extend the full depth of the diaphragm, the wall shear should be divided by the diaphragm width to determine the load per foot to the collector. The collector forces and connection requirements can then be determined by multiplying the load per foot to the collector by the collector length from its end to the location being analyzed.

C10.3.6.2 Diaphragm/Frame Shear Transfer

The floor and roof diaphragm must be adequately connected to the steel frames to provide a load path for the shears in the diaphragm to be delivered to the frames.

After calculating the shear force at the frame being analyzed, this force should be divided by the depth of the diaphragm to determine the shear per foot transfer requirement to the collector and frames. Collector forces can be determined by multiplying the shear per foot by the length from the end of the collector to the location being analyzed.

C10.3.6.3 Anchorage for Normal Forces

Walls that are not well anchored to the diaphragms may separate from the remainder of the structure and collapse during an earthquake. If these walls are bearing walls, partial floor collapse may result. The hazard amplifies with the height above the building base, and is affected by the soil type and the type and configuration of the walls and/or diaphragms.

Several guidelines for the evaluation of wall anchorage are provided in FEMA 178 (BSSC, 1992a). First, cross-grain tension can lead to abrupt brittle failures in wood ledgers; this condition should be eliminated. Second, wood diaphragms should be directly anchored to the walls for out-of-plane loading. Third, steel anchors should be utilized, and well developed into the diaphragm to achieve adequate capacity and ductility. Finally, anchorage from the floors or roof into the walls should have sufficient spacing, strength, and stability. For further explanation of these statements, refer to FEMA 178.

C10.3.6.4 Girder-Wall Connections

Where girder-wall connections are a primary part of the out-of-plane load path, the anchorage into the wall should be ductile. If the girder rests on a corbel, the bearing length should be adequate to accommodate expected motions. Where precast girders are welded to column corbels, unintended frame action may attract high seismic forces.

C10.3.6.5 Precast Connections

Precast concrete frames without shear walls must not be addressed under Simplified Rehabilitation (see *Guidelines* Table 10-1). For precast frames that are braced by concrete shear walls, the interconnections of elements that serve as the chords, ties, and collectors must be similar. These connections should be evaluated to determine whether they are adequate. Special consideration must be given to their as-built condition, since they are susceptible to failures induced by thermal stresses and corrosion.

C10.3.6.6 Wall Panels and Cladding

The connections between wall panels or cladding and the structural framing are important for preventing damage to both elements. Typically, cladding is not constructed integrally with the framing but is added afterward, so the connection often forms a potential weak link. The cladding, which is not designed as part of the lateral-force-resisting system, should be isolated so as not to be damaged by building drifts, yet anchored to prevent falling out under strong shaking. Precast concrete wall panels can themselves be much stiffer than the lateral-force-resisting system in a moment frame building; thus, if rigidly attached to the frame they can actually attract forces and route them through unintended load paths.

Systematic Rehabilitation Analysis Procedures may be beneficial for determining the actual expected building drifts. (FEMA 178 [BSSC, 1992a], Section 8.6.2.)

C10.3.6.7 Light Gage Metal, Plastic or Cementitious Roof Panels

The connections between flexible roof diaphragms and the structural framing are important for developing a building's load path. Typically, these types of roofs are not constructed integrally with the framing (as opposed to a concrete slab or deck and fill), so the connection often forms a potential weak link. (FEMA 178 [BSSC, 1992a], Section 8.6.1.)

The forces in the diaphragm can typically be determined by noncomputerized analysis using tributary areas. The existing connections should be checked for the forces developed.

C10.3.6.8 Mezzanine Connections

It is very common for mezzanines to lack a lateral-force-resisting system. If the mezzanine lacks bracing elements or is not adequately connected to walls or framing capable of adequately bracing the mezzanine, the mezzanine can be fully isolated and investigated as a separate structure. Lateral-force-resisting elements must be present in both directions to provide bracing.

C10.3.7 Foundations and Geologic Hazards

C10.3.7.1 Anchorage to Foundations

For FEMA 178 evaluation statements to be true, steel columns and wood posts must be positively attached to the foundation. Concrete columns are required to have longitudinal steel doweled into the foundation. Similarly, doweled reinforcing for masonry and concrete walls is required. It is also required that wood walls be anchored with bolts or drilled anchors. The ends of shear walls must be substantially anchored into the building foundation to resist overturning.

Where the bases of steel and wood columns are exposed, it is relatively easy to identify whether they are anchored to the foundation. In the case of concrete columns or walls it may be very difficult to determine, in the absence of drawings, whether there are foundation dowels. Generally, it is relatively more important that columns—particularly wood and steel columns—be anchored to the foundation to prevent movement during seismic shaking and potential loss of vertical support, than that walls be so anchored. It is improbable that concrete columns or walls would be displaced to the point of causing a vertical load-carrying deficiency during an earthquake due to lack of dowels into a footing. It also seems unreasonable to require URM walls to be anchored to the foundation, whereas reinforced masonry or concrete walls would be required to be doweled. With respect to wood frame walls and foundation anchorage, it is not generally considered to be a life safety hazard if a wood frame building is not anchored to its foundation. Judgment should be exercised in determining the need for the type of anchorage implied by the FEMA 178 provision. If lateral loads are resisted by a relatively few, highly stressed elements, such anchorage may be important. However, in buildings where there are a substantial number of walls resisting loads at relatively low stress, anchorage to the footings may not be necessary for the Life Safety Performance Level.

When anchorage requirements for vertical elements are determined to be necessary because of high stresses or relatively few elements, and the repairs required to do so are costly and/or intrusive, Systematic Rehabilitation measures are recommended. This is due to the fact that the more detailed Analysis Procedures may allow reduction in forces and, in some cases, justification that anchorage is not required, especially in the case of anchorage at ends of shear walls where some rocking due to lack of tension restraint at the ends of walls may be analytically justified. (FEMA 178 [BSSC, 1992a], Sections 8.4.1 - 8.4.7.)

C10.3.7.2 Condition of Foundations

The FEMA 178 evaluation statements relate to signs of excessive foundation movement or of deterioration due to corrosion or other material conditions. The intention is to verify that the foundation has performed adequately under prior loading, which normally includes dead loads, live loads, wind, and, in some cases, previous earthquakes. If this performance has been satisfactory there is less reason to be concerned over future performance during earthquakes. Similarly, with respect to deterioration of foundation elements and materials, if no signs of degradation are present it is reasonable to assume that the foundations will remain in serviceable condition.

The procedure for investigating the condition of existing foundations in FEMA 178 is essentially one of visual inspection. The difficulty is that both the deterioration of existing elements and materials problems are not always readily observable. In some cases excavation can be used to expose existing piles or pier footings for investigation. Some conditions can be easily identified, including spalling of concrete due to corrosion of rebar, or discoloration due to sulfate attack. With respect to settlement or distress due to loads in existing foundations, some measurements may be helpful. It is expected that building foundations, particularly shallow spread footings, will undergo some movement during the life of a structure; however, excessive differential settlements can cause distress to structural elements that are needed to resist seismic loading. For example, differential settlement in steel frames can actually cause yielding of moment connections. Angular distortions that exceed 0.25% to 0.50%, depending on the type of construction, should be investigated using more detailed field investigation and, probably, Systematic Rehabilitation. In addition to measuring changes in relative elevations, observations can be made of brittle concrete or masonry elements to identify cracking.

For foundations with signs of excessive distress—due to either service loading or material conditions—detailed investigations, including Systematic Rehabilitation, are warranted. These cases, however, will be unusual because building foundations generally perform well and should not be subject to intense scrutiny, unless there are signs of significant deterioration or distress. (FEMA 178 [BSSC, 1992a], Sections 9.1.1 - 9.1.2.)

C10.3.7.3 Overturning

If a building is sufficiently short compared to its base dimension, overturning effects may be neglected. The criteria in FEMA 178 (BSSC, 1992a) are related to anticipated seismicity of the area by the velocity-rated acceleration factor. Buildings in areas of relatively low seismicity may be more slender and still not require consideration of overturning effects.

If the geometric requirement (base-to-height ratio) of FEMA 178 (BSSC, 1992a) is exceeded, simplified calculations are required. For shallow foundations, if bearing pressures under total gravity loads plus earthquake loads do not exceed two times the allowable static bearing pressures, the foundation is considered adequate for overturning. For deep foundations, the total load may not exceed the ultimate vertical capacity of the pile or piers.

If the simplified calculations are required, FEMA 178 does not provide guidance on the determination of the allowable capacity of shallow foundations nor the ultimate capacity of deep foundations. In some cases this information may be available from previous soils reports or from consultation with a qualified geotechnical engineer. Building failures from excessive foundation loading have very seldom been observed in past earthquakes. Additionally, some amount of foundation yielding and movement tends to reduce the forces transmitted to the superstructure. In this sense, inelastic behavior in the foundation is considered desirable.

The type of mitigative action required to correct overturning problems of foundations is generally very expensive. For this reason, it is strongly recommended that Systematic Rehabilitation be used for evaluation, design, and construction of mitigation measures for overturning. Chapter 4 of the *Guidelines* provides procedures for estimating foundation stiffnesses and capacities, for use in analyses to evaluate foundation performance more realistically. More realistic evaluation and design methods slightly increase engineering cost, but in cases such as this are likely to reduce construction costs considerably. (FEMA 178 [BSSC, 1992a], Section 9.2.1.)

C10.3.7.4 Lateral Loads

Lateral loads at the foundation level are transferred to the supporting soil by friction or passive pressure on the sides and bottoms of foundation elements. FEMA 178 evaluation statements require that these elements be capable of transferring lateral loads. Specific guidance on allowable horizontal loads or pressures is not provided. Ties between foundation elements are also required to be “adequate.” Also, building sites where significant difference in grade exists across a building site must account for lateral earthquake forces due to soil pressures on foundation walls.

FEMA 178 provides only a very qualitative assessment of lateral load transfer. Judgment should be used. For buildings in which the lateral load is transferred to the supporting soil in relatively few locations that are not generally tied to the rest of the structure, some conservatism is warranted. Concrete slabs on grade are most often adequate to tie foundations together as a unit. Experience in past earthquakes does not indicate that sliding, or lateral bearing, failure causes life safety problems in the absence of some differential vertical or horizontal permanent ground displacement—due to liquefaction, lateral spreading, or some other geologic site hazard.

C10.3.7.5 Geologic Site Hazards

FEMA 178 includes evaluation statements for liquefaction, slope failure, and surface fault rupture, which identify cases requiring detailed investigation.

C10.3.8 Evaluation of Materials and Conditions

C10.3.8.1 General

Techniques used in this evaluation step may range from simple visual inspection through sample removal and destructive testing in a laboratory. Visual inspection includes direct viewing techniques, noninvasive techniques (e.g., temporary removal of coverings, use of a fiberscope), or invasive exploration, which requires repairs to finishes after access and completion of inspection. Nondestructive and destructive testing techniques used are specific to the material type (e.g., wood, steel). Typical methods and their application are addressed in Chapters 5 through 8. Extension of visual inspection techniques includes the grading of wood lumber type and quality of construction, and evaluation of seismic deficiencies using FEMA 178 (BSSC, 1992a).

Recovery of original design and construction documentation is also necessary, as this information generally defines original component sizes, material strengths, connection configuration, and overall dimensions. The design professional shall conduct research to accumulate available construction documents, including interviews with the original architect-engineer and contractor. If the data do not exist and the original design and construction team is not known, it is necessary to prepare as-built layouts of the existing structural system and to determine material properties for the affected components.

Default material properties that may be used for guidance are included in Chapters 5 through 8 of the

Guidelines; these values would be verified as representative through a limited amount of testing of samples from existing components. Sampling and test methods for determining materials strength and other properties are similarly contained in Chapters 5 through 8. In general, the following minimum numbers of tests should be performed (the amount of data already known about the structure and quality of construction may reduce that number).

- When drawings and data on original construction exist, material variability is low (less than 25%), building height is two stories or less, and plan area is less than 2,000 square feet, three tests may be performed on random samples from each primary component type affected.
- When only limited drawings or information exist, the deficiency or damage is comprehensive, material properties have significant variance, or the building height and plan area exceed two stories and 2,000 square feet, six tests should be performed on random samples removed from each primary component type affected.

It is expected that additional tests will be planned by the design professional to address any abnormal conditions or deficiencies.

The extent of the deficiency or damage shall be determined through a combination of visual inspection and testing. The design professional shall establish the condition of in-place materials and affected structural systems as part of the evaluation process. Similarly, any constraints associated with the rehabilitation process—such as reinforcing material fit-up, access for strengthening, temporary abandonment of the building, and removal of coverings with historical value—shall also occur at this stage. Information gained in the evaluation phase shall be used in the analysis and design of rehabilitation measures. If possible, the scope of rehabilitation shall be reviewed with the client, owner, code official, and other involved parties (e.g., contractor) at the building site to ensure that all rehabilitation goals are met.

C10.3.8.2 Condition of Wood

The condition of the wood in a structure has a direct relationship to its performance in a seismic event. Wood that is split, rotten, or has insect damage may have a very low capacity to resist loads imposed by earthquakes. Structures with wood elements depend to a large extent on the connections between members. If the wood at a bolted connection is split, the connection will possess only a fraction of the capacity of a similar connection in sound wood.

A preliminary analysis of the structure will generally lead to an indication of the critical connections and members that are part of the lateral-load-resisting system for the structure. These members and connections are the logical areas to inspect for possible deterioration problems. The wood members should be examined by exposing a representative sample of locations and visually examining and probing the wood with an awl or small drill to determine the condition and extent of any rot or decay. (FEMA 178 [BSSC, 1992a], Section 3.5.1.)

C10.3.8.3 Overdriven Fasteners

Fasteners connecting structural panels to the framing are supposed to be driven flush with—but should not penetrate—the surface of the sheathing.

For structures built prior to the wide use of nailing guns (pre-1970), the problem is generally not present. More recent projects are often constructed with alternative fasteners, such as staples, T-nails, clipped nail heads, or cooler nails, installed with pneumatic nail guns and often overdriven, completely penetrating one or more panel plys. This effectively reduces the shear capacity of the fastener. Nail shank diameter should also be checked for conformance with the common nail value, which is the basis for the shear values established in most reference documents.

The overdriven fasteners can be evaluated by comparing the length of the fastener in the panel to the thickness of the panel and reducing the capacity of the panel by the same ratio. (FEMA 178 [BSSC, 1992a], Section 3.5.2.)

C10.3.8.4 Condition of Steel

Environmental effects over prolonged periods of time may lead to deterioration of elements of steel lateral-force-resisting frames. Deterioration, in the form of rusting or corrosion, can significantly reduce the member cross sections, with a corresponding reduction in capacity. Such deterioration must be considered in the seismic evaluation.

Appropriate estimates of the capacity reduction that has occurred must be based on the extent of field investigation performed. If significant deterioration is observed, more extensive field work may be justified. Estimates of the deterioration in other elements that were not specifically evaluated may be required.

In addition to repair of damage, the causes of deterioration must be determined through investigation, and eliminated to protect the steel in the future. The demands on the existing elements can be reduced by the addition of braced bays, shear wall panels, or base isolation.

Systematic Rehabilitation Analytical Procedures should be used in tall and/or irregular buildings to determine the expected frame demands. (FEMA 178 [BSSC, 1992a], Section 3.5.3.)

C10.3.8.5 Condition of Concrete

Damaged or deteriorated material may not be readily observable. Visual inspection should be conducted.

Visual inspection of the material may be adequate if the damage is not severe and the intent is to patch and repair the distressed region. Where the existing material will remain without modification, appropriate tests should be conducted to determine the usable strength.

In general, the most straightforward Simplified Rehabilitation Method solution would be to identify the causes of the condition and define corrective methods to prevent the deterioration from continuing, and to remove and replace the deteriorated material using appropriate repair techniques (see ACI publications). (FEMA 178 [BSSC, 1992a], Sections 3.5.4, 3.5.5, 3.5.8.)

C10.3.8.6 Post-Tensioning Anchors

Corrosion in post-tensioning anchors can lead to failure of gravity systems if ground shaking causes a release or slip of prestressing strands. Coil anchors (with or without corrosion) have performed poorly under cyclic loads.

The material around the anchors should be sound and capable of providing adequate encasement of the anchor. Inspection of the anchors should be visual, and may involve chipping away surface material if there is evidence of internal corrosion or deterioration. (FEMA 178 [BSSC, 1992a], Section 3.5.5.)

C10.3.8.7 Quality of Masonry

If the masonry walls do not pass the FEMA 178 evaluation statements, one alternative is to discount the strength of sections of walls that do not pass the calculations.

The ASTM standards on mortar, sponsored by ASTM Committee C-12, provide information on repointing mortar. In order to restore the strength of the wall to its initial condition, all of the eroded mortar must be replaced by repointing. In the event that this is not practical, the wall should be tested in accordance with Appendix C of FEMA 178 to determine the allowable stresses that may be used. (FEMA 178 [BSSC, 1992a], Sections A4, Sections 3.5.9, 3.5.10, and 3.5.11.)

C10.4 Amendments to FEMA 178

Several amendments to FEMA 178 (BSSC, 1992a) have been developed in Section 10.4 of the *Guidelines*. They are based on deficiencies observed as a result of significant earthquakes that have occurred since the publication of FEMA 178. The eight new deficiencies are presented in the same style as in FEMA 178; the format includes a true/false evaluation statement, a paragraph of commentary to identify the concern, and a suggested procedure to follow if the evaluation statement is found to be false. The new amendments are covered in *Guidelines* Section 10.4 and are included in the complete list of FEMA 178 (BSSC, 1992a) deficiencies, including the amendments, in Section C10.5 of this *Commentary.*

C10.5 FEMA 178 Deficiency Statements

This *Commentary* section provides a complete list of all FEMA 178 (BSSC, 1992a) deficiency evaluation statements, as well as the eight new potential deficiencies listed in Section 10.4 of the *Guidelines*, presented in a logical, combined order.

C10.5.1 Building Systems

C10.5.1.1 Load Path

The structure contains a complete load path, for seismic force effects from any horizontal direction, that serves to transfer the inertial forces from the mass to the foundation. (FEMA 178 [BSSC, 1992a], Section 3.1.)

C10.5.1.2 Redundancy

The structure will remain laterally stable after the failure of any single element. (FEMA 178 [BSSC, 1992a], Section 3.2.)

C10.5.1.3 Vertical Irregularities

A. Weak Story

Visual observation or a Quick Check indicates that there are no significant strength discontinuities in any of the vertical elements in the lateral-force-resisting system; the story strength at any story is not less than 80% of the strength of the story above. (FEMA 178 [BSSC, 1992a], Section 3.3.1.)

B. Soft Story

Visual observation or a Quick Check indicates that there are no significant stiffness discontinuities in any of the vertical elements in the lateral-force-resisting system; the lateral stiffness of a story is not less than 70% of that in the story above or less than 80% of the average stiffness of the three stories above. (FEMA 178 [BSSC, 1992a], Section 3.3.2.)

C. Geometry

There are no significant geometrical irregularities; there are no setbacks (i.e., no changes in horizontal dimension of the lateral-force-resisting system of more than 30% in a story relative to the adjacent stories). (FEMA 178 [BSSC, 1992a], Section 3.3.3.)

D. Mass

There are no significant mass irregularities; there is no change of effective mass of more than 50% from one story to the next, excluding light roofs. (FEMA 178 [BSSC, 1992a], Section 3.3.4.)

E. Vertical Discontinuities

All shear walls, infilled walls, and frames are continuous to the foundation. (FEMA 178 [BSSC, 1992a], Section 3.3.5.)

C10.5.1.4 Plan Irregularities Creating Torsion

The lateral-force-resisting elements form a well-balanced system that is not subject to significant torsion. Significant torsion will be taken as any condition where the distance between the story center of rigidity and the story center of mass is greater than 20% of the width of the structure in either major plan dimension. (FEMA 178 [BSSC, 1992a], Section 3.3.6.)

C10.5.1.5 Adjacent Buildings

There is no immediately adjacent structure that is less than half as tall or has floors/levels that do not match those of the building being evaluated. A neighboring structure is considered to be “immediately adjacent” if it is within two inches times the number of stories away from the building being evaluated. (FEMA 178 [BSSC, 1992a], Section 3.4.)

C10.5.1.6 Lateral Load Path at Pile Caps

Pile caps are capable of transferring lateral and overturning forces between the structure and individual piles in the pile group.

C10.5.1.7 Deflection Compatibility

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.

C10.5.2 Moment Frames

C10.5.2.1 Steel Moment Frames

A. Drift Check

The building satisfies the Quick Check of the frame drift. (FEMA 178 [BSSC, 1992a], Section 4.2.1.)

B. Frame Concerns

Compact Members. All moment frame elements meet the compact section requirements of the basic AISC documents (AISC, 1986 and 1989). (FEMA 178 [BSSC, 1992a], Section 4.2.2.)

Beam Penetrations. All openings in beam webs have a depth less than one-quarter of the beam depth and are located in the center half of the beams. (FEMA 178 [BSSC, 1992a], Section 4.2.3.)

Out-of-Plane Bracing. Beam-column joints are braced out-of-plane. (FEMA 178 [BSSC, 1992a], Section 4.2.9.)

C. Strong Column-Weak Beam

In areas of high seismicity (A_v greater than or equal to 0.2), at least one-half of the joints are strong column-weak beam (33% on every line of moment frame). Roof frame joints need not be considered. (FEMA 178 [BSSC, 1992a], Section 4.2.8.)

D. Connections

Moment Connections. All beam-column connections in the lateral-force-resisting moment frame have full-penetration flange welds and a bolted or welded web connection. (FEMA 178 [BSSC, 1992a], Section 4.2.4.)

Column Splices. In areas of high seismicity (A_v greater than or equal to 0.2), all column splice details of the moment-resisting frames include connection of both flanges and the web. (FEMA 178 [BSSC, 1992a], Section 4.2.5.)

Joint Webs. All web thicknesses within joints of moment-resisting frames meet the AISC criteria for web shear (AISC, 1986 and 1989). (FEMA 178 [BSSC, 1992a], Section 4.2.6.)

Girder Flange Continuity Plates. There are girder flange continuity plates at joints. (FEMA 178 [BSSC, 1992a], Section 4.2.7.)

Moment-Resisting Connections. All moment connections are able to develop the strength of the adjoining members or panel zones.

C10.5.2.2 Concrete Moment Frames

A. Quick Checks, Frame, and Nonductile Detail Concerns

Shearing Stress Check. The building satisfies the Quick Check of the average shearing stress in the columns. (FEMA 178 [BSSC, 1992a], Section 4.3.1.)

Drift Check. The building satisfies the Quick Check of story drift. (FEMA 178 [BSSC, 1992a], Section 4.3.2.)

Prestressed Frame Elements. The lateral-load-resisting frames do not include any prestressed or post-tensioned elements. (FEMA 178 [BSSC, 1992a], Section 4.3.3.)

Joint Eccentricity. There are no eccentricities larger than 20% of the smallest column plan dimension between girder and column centerlines. (FEMA 178 [BSSC, 1992a], Section 4.3.4.)

No Shear Failures. The shear capacity of frame members is greater than the moment capacity. (FEMA 178 [BSSC, 1992a], Section 4.3.5.)

Strong Column-Weak Beam. The moment capacity of the columns is greater than that of the beams. (FEMA 178 [BSSC, 1992a], Section 4.3.6.)

Stirrup and Tie Hooks. The beam stirrups and column ties are anchored into the member cores with hooks of 135 degrees or more. (FEMA 178 [BSSC, 1992a], Section 4.3.7.)

Column-Tie Spacing. Frame columns have ties spaced at $d/4$ or less throughout their length and at $8d_b$ or less at all potential plastic hinge regions. (FEMA 178 [BSSC, 1992a], Section 4.3.8.)

Column-Bar Splices. All column-bar lap splice lengths are greater than $35d_b$ long and are enclosed by ties spaced at $8d_b$ or less. (FEMA 178 [BSSC, 1992a], Section 4.3.9.)

Beam Bars. At least two longitudinal top and two longitudinal bottom bars extend continuously throughout the length of each frame beam. At least 25% of the steel provided at the joints for either positive or negative moment is continuous throughout the members. (FEMA 178 [BSSC, 1992a], Section 4.3.10.)

Beam-Bar Splices. The lap splices for the longitudinal beam reinforcing are located within the center half of the member lengths and not in the vicinity of potential plastic hinges. (FEMA 178 [BSSC, 1992a], Section 4.3.11.)

Stirrup Spacing. All beams have stirrups spaced $d/2$ or less throughout their length and at $8d_b$ or less at potential hinge locations. (FEMA 178 [BSSC, 1992a], Section 4.3.12.)

Beam Truss Bars. Bent-up longitudinal steel is not used for shear reinforcement. (FEMA 178 [BSSC, 1992a], Section 4.3.13.)

Joint Reinforcing. Column ties extend at their typical spacing through all beam-column joints at exterior columns. (FEMA 178 [BSSC, 1992a], Section 4.3.14.)

Flat Slab Frames. The system is not a frame consisting of a flat slab/plate without beams. (FEMA 178 [BSSC, 1992a], Section 4.3.15.)

B. Precast Moment Frames

The lateral loads are not resisted by precast concrete frame elements. (FEMA 178 [BSSC, 1992a], Section 4.4.1.)

C10.5.2.3 Frames Not Part of the Lateral-Force-Resisting System

A. Short Captive Columns

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.

C10.5.3 Shear Walls

C10.5.3.1 Cast-in-Place Concrete Shear Walls

A. Shearing Stress Check

The building satisfies the Quick Check of the shearing stress in the shear walls. (FEMA 178 [BSSC, 1992a], Section 5.1.1.)

B. Overturning

All shear walls have h_w/l_w ratios less than four to one. (FEMA 178 [BSSC, 1992a], Section 5.1.2.)

C. Coupling Beams

The stirrups in all coupling beams over means of egress are spaced at $d/2$ or less and are anchored into the core with hooks of 135 degrees or more. (FEMA 178 [BSSC, 1992a], Section 5.1.3.)

D. Boundary Element Detailing

Column Splices. Steel column splice details in shear wall boundary elements can develop the tensile strength of the column. (FEMA 178 [BSSC, 1992a], Section 5.1.4.)

Wall Connections. There is positive connection between the shear walls and the steel beams and columns. (FEMA, 178, Section 5.1.5.)

Confinement Reinforcing. For shear walls with h_w/l_w greater than 2.0, the boundary elements are confined with spirals or ties with spacing less than $8d_b$. (FEMA 178 [BSSC, 1992a], Section 5.1.6.)

E. Wall Reinforcement

Reinforcing Steel. The area of reinforcing steel for concrete walls is greater than 0.0025 times the gross area of the wall along both the longitudinal and transverse axes, and the maximum spacing of reinforcing steel is 18 inches. (FEMA 178 [BSSC, 1992a], Section 5.1.7.)

Reinforcing at Openings. There is special wall reinforcement around all openings. (FEMA 178 [BSSC, 1992a], Section 5.1.8.)

Shear Stress Check. The building satisfies the Quick Check of the shearing stress in wood shear walls. (FEMA 178 [BSSC, 1992a], Section 5.6.1.)

Openings. Walls with garage doors or other large openings are braced with plywood shear walls, or supported by adjacent construction through substantial positive ties. (FEMA 178 [BSSC, 1992a], Section 5.6.2.)

Wall Requirements. All walls supporting tributary areas of 24 to 100 square feet per foot of wall are plywood-sheathed with proper nailing, or rod-braced, and have a height-to-depth ratio of one to one or less, or have properly detailed and constructed hold-downs. (FEMA 178 [BSSC, 1992a], Section 5.6.3.)

Cripple Walls. All exterior cripple walls below the first floor level are braced to the foundation with shear elements. (FEMA 178 [BSSC, 1992a], Section 5.6.4.)

C10.5.3.2 Precast Concrete Shear Walls

A. Panel-to-Panel Connections

Adjacent wall panels are not connected by welded steel inserts. (FEMA 178 [BSSC, 1992a], Section 5.2.1.)

B. Wall Openings

Openings constitute less than 75% of the length of any perimeter wall, with the wall piers having h_w/l_w ratios of less than 2.0. (FEMA 178 [BSSC, 1992a], Section 5.2.2.)

C. Collectors

Wall elements with openings larger than a typical panel at a building corner are connected to the remainder of the wall with collector reinforcing. (FEMA 178 [BSSC, 1992a], Section 5.2.3.)

C10.5.3.3 Masonry Shear Walls

A. Reinforcing in Masonry Walls

In areas of high seismicity (A_v greater than or equal to 0.2): (1) the total vertical and horizontal reinforcing steel in reinforced masonry walls is greater than 0.002 times the gross area of the wall, with a minimum of 0.0007 in either of the two directions; (2) the spacing of reinforcing steel is less than 48 inches; and (3) all vertical bars extend to the top of the walls. (FEMA 178 [BSSC, 1992a], Section 5.3.2.)

B. Shearing Stress Check

The building satisfies the Quick Check of the shearing stress in the reinforced masonry shear walls. (FEMA 178 [BSSC, 1992a], Section 5.3.1.)

C. Reinforcing at Openings

All wall openings that interrupt rebar have trim reinforcing on all sides. (FEMA 178 [BSSC, 1992a], Section 5.3.3.)

D. Unreinforced Masonry Shear Walls

Shearing Stress Check. The building satisfies the Quick Check of the shearing stress in the unreinforced masonry shear walls. (FEMA 178 [BSSC, 1992a], Section 5.4.1.)

Masonry Lay-up. Filled collar joints of multiwythe masonry walls have negligible voids. (FEMA 178 [BSSC, 1992a], Section 5.4.2.)

E. Proportions, Solid Walls

Proportions. In areas of high seismicity (A_v greater than or equal to 0.2), the height-to-thickness ratio of the unreinforced masonry wall panels is as follows:

One-story building $h_w/t < 14$

Multistory building

Top story $h_w/t < 9$

Other stories $h_w/t < 20$

(FEMA 178 [BSSC, 1992a], Section 5.5.1.)

Solid Walls. The unreinforced masonry infill walls are not of cavity construction. (FEMA 178 [BSSC, 1992a], Section 5.5.2.)

F. Infill Walls

The unreinforced masonry infill walls are continuous to the soffits of the frame beams. (FEMA 178 [BSSC, 1992a], Section 5.5.3.)

C10.5.3.4 Shear Walls in Wood Frame Buildings

A. Shear Stress Check

The building satisfies the Quick Check of the shearing stress in wood shear walls. (FEMA 178 [BSSC, 1992a], Section 5.6.1.)

B. Openings

Walls with garage doors or other large openings are braced with plywood shear walls or supported by adjacent construction through substantial positive ties. (FEMA 178 [BSSC, 1992a], Section 5.6.2.)

C. Wall Requirements

All walls supporting tributary areas of 24 to 100 square feet per foot of wall are plywood sheathed with proper nailing, or rod-braced, and have a height-to-depth ratio of one to one or less, or have properly detailed and constructed hold-downs. (FEMA 178 [BSSC, 1992a], Section 5.6.3.)

D. Cripple Walls

All exterior cripple walls below the first floor level are braced to the foundation with shear elements. (FEMA 178 [BSSC, 1992a], Section 5.6.4.)

E. Narrow Wood Shear Walls

Narrow wood shear walls with an aspect ratio greater than two to one do not resist forces developed in the building.

F. Stucco Shear Walls

Multistory buildings do not rely on exterior stucco walls as the primary lateral-force-resisting system.

G. Gypsum Wallboard or Plaster Shear Walls

Interior gypsum wallboard or plaster is not being used for shear walls on buildings over one story in height.

C10.5.4 Steel Braced Frames

C10.5.4.1 Stress Check

The building satisfies the Quick Check of the stress in the diagonals. (FEMA 178 [BSSC, 1992a], Section 6.1.1.)

C10.5.4.2 Stiffness of Diagonals

A. Stiffness of Diagonals

All diagonal elements required to carry compression have Kl/r ratios less than 120. (FEMA 178 [BSSC, 1992a], Section 6.1.2.)

B. Tension-only Braces

Tension-only braces are not used as the primary diagonal bracing elements in structures over two stories in height. (FEMA 178 [BSSC, 1992a], Section 6.1.3.)

C10.5.4.3 Chevron or K-Bracing

The bracing system does not include chevron, V-, or K-braced bays. (FEMA 178 [BSSC, 1992a], Section 6.1.4.)

C10.5.5 Diaphragms

C10.5.5.1 Plan Irregularities: Re-entrant Corners

There is significant tensile capacity at re-entrant corners or other locations of plan irregularities. (FEMA 178 [BSSC, 1992a], Section 7.1.1.)

C10.5.5.2 Crossties

There are continuous crossties between diaphragm chords. (FEMA 178 [BSSC, 1992a], Section 7.1.2.)

C10.5.5.3 Diaphragm Openings

A. Reinforcing at Openings

There is reinforcing around all diaphragm openings that are larger than 50% of the building width in either major plan dimension. (FEMA 178 [BSSC, 1992a], Section 7.1.3.)

B. Openings at Shear Walls

Diaphragm openings immediately adjacent to the shear walls constitute less than 25% of the wall length, and the available length appears sufficient. (FEMA 178 [BSSC, 1992a], Section 7.1.4.)

C. Openings at Braced Frames

Diaphragm openings immediately adjacent to the braced frames extend less than 25% of the length of the bracing. (FEMA 178 [BSSC, 1992a], Section 7.1.5.)

D. Openings at Exterior Masonry Shear Walls

Diaphragm openings immediately adjacent to exterior masonry walls are no more than eight feet long. (FEMA 178 [BSSC, 1992a], Section 7.1.6.)

C10.5.5.4 Sheathing

None of the diaphragms consist of straight sheathing or have span-to-depth ratios greater than two to one. (FEMA 178 [BSSC, 1992a], Section 7.2.1.)

C10.5.5.5 Unblocked Diaphragms

Unblocked wood panel diaphragms consist of horizontal spans less than 40 feet and have span-to-depth ratios less than or equal to three to one. (FEMA 178 [BSSC, 1992a], Section 7.2.3.)

C10.5.5.6 Spans

All diaphragms with spans greater than 24 feet have plywood or diagonal sheathing. Wood commercial and industrial buildings may have rod-braced systems. (FEMA 178 [BSSC, 1992a], Section 7.2.2.)

C10.5.5.7 Span-to-Depth Ratio

If the span-to-depth ratios of wood diaphragms are greater than three to one, there are nonstructural walls connected to all diaphragm levels at less than 40-foot spacing. (FEMA 178 [BSSC, 1992a], Section 7.2.4.)

C10.5.5.8 Diaphragm Continuity

None of the diaphragms are composed of split-level floors or, in wood commercial or industrial buildings, have expansion joints. (FEMA 178 [BSSC, 1992a], Section 7.2.5.)

C10.5.5.9 Chord Continuity

All chord elements are continuous, regardless of changes in roof elevation. (FEMA 178 [BSSC, 1992a], Section 7.2.6.)

C10.5.6 Connections

C10.5.6.1 Diaphragm/Wall Shear Transfer

A. Transfer to Shear Walls

Diaphragms are reinforced for transfer of loads to the shear walls. (FEMA 178 [BSSC, 1992a], Section 8.3.1.)

B. Topping Slab to Walls and Frames

Reinforced concrete topping slabs that interconnect the precast concrete diaphragm elements are doweled into the shear wall or frame elements. (FEMA 178 [BSSC, 1992a], Section 8.3.3.)

C10.5.6.2 Diaphragm/Frame Shear Transfer

A. Transfer to Steel Frames

The method used to transfer diaphragm shears to the steel frames is approved for use under lateral loads. (FEMA 178 [BSSC, 1992a], Section 8.3.2.)

B. Topping Slab to Walls and Frames

Reinforced concrete topping slabs that interconnect the precast concrete diaphragm elements are doweled into the shear wall or frame elements. (FEMA 178 [BSSC, 1992a], Section 8.3.3.)

C10.5.6.3 Anchorage for Normal Forces

A. Wood Ledgers

The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers. (FEMA 178 [BSSC, 1992a], Section 8.2.1.)

B. Wall Anchorage

The exterior concrete or masonry walls are anchored to each of the diaphragm levels for out-of-plane loads. (FEMA 178 [BSSC, 1992a], Section 8.2.2.)

C. Masonry Wall Anchors

Wall anchorage connections are steel anchors or straps that are developed into the diaphragm. (FEMA 178 [BSSC, 1992a], Section 8.2.3.)

D. Anchor Spacing

The anchors from the floor and roof systems into exterior masonry walls are spaced at four feet or less. (FEMA 178 [BSSC, 1992a], Section 8.2.4.)

E. Tilt-up Walls

Precast bearing walls are connected to the diaphragms for out-of-plane loads; steel anchors or straps are embedded in the walls and developed into the diaphragm. (FEMA 178 [BSSC, 1992a], Section 8.2.5.)

F. Panel-Roof Connection

There are at least two anchors from each precast wall panel into the diaphragm elements. (FEMA 178 [BSSC, 1992a], Section 8.2.6.)

G. Stiffness of Wall Anchors

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 inch larger than the bolt diameter.

C10.5.6.4 Girder-Wall Connections

A. Girders

Girders that are supported by walls or pilasters have special ties to secure the anchor bolts. (FEMA 178 [BSSC, 1992a], Section 8.5.1.)

B. Corbel Bearing

If the frame girders bear on column corbels, the length of bearing is greater than three inches. (FEMA 178 [BSSC, 1992a], Section 8.5.2.)

C. Corbel Connections

The frame girders are not supported on corbels with welded elements. (FEMA 178 [BSSC, 1992a], Section 8.5.3.)

C10.5.6.5 Braced Frame Connections

A. Concentric Joints

All the diagonal braces frame into the beam-column joints concentrically. (FEMA 178 [BSSC, 1992a], Section 6.1.5.)

B. Connection Strength

All the brace connections are able to develop the yield capacity of the diagonals. (FEMA 178 [BSSC, 1992a], Section 6.1.6.)

C. Column Splices

All column splice details of the braced frames can develop the column yield capacity. (FEMA 178 [BSSC, 1992a], Section 6.1.7.)

C10.5.6.6 Precast Connections

For buildings with concrete shear walls, the connection between precast frame elements—such as chords, ties, and collectors—in the lateral-force-resisting system can develop the capacity of the connected members. (FEMA 178 [BSSC, 1992a], Section 4.4.2.)

C10.5.6.7 Wall Panels

All wall panels (metal, fiberglass, or cementitious) are properly connected to the wall framing. (FEMA 178 [BSSC, 1992a], Section 8.6.2.)

C10.5.6.8 Light Gage Metal, Plastic, or Cementitious Roof Panels

All light gage metal, plastic, or cementitious roof panels are properly connected to the roof framing at not more than 12 inches on center. (FEMA 178 [BSSC, 1992a], Section 8.6.1.)

C10.5.7 Foundations and Geologic Hazards

C10.5.7.1 Anchorage of Vertical Components to Foundations

A. Steel Columns

The columns in the lateral-force-resisting frames are substantially anchored to the building foundation. (FEMA 178 [BSSC, 1992a], Section 8.4.1.)

B. Concrete Columns

All longitudinal column steel is doweled in the foundation. (FEMA 178 [BSSC, 1992a], Section 8.4.2.)

C. Wood Posts

There is positive connection of wood posts to the foundation and the elements being supported. (FEMA 178 [BSSC, 1992a], Section 8.4.3.)

D. Wall Reinforcing

All vertical wall reinforcing is doweled into the foundation. (FEMA 178 [BSSC, 1992a], Section 8.4.4.)

E. Shear-Wall-Boundary Columns

The shear wall columns are substantially anchored to the building foundation. (FEMA 178 [BSSC, 1992a], Section 8.4.5.)

F. Wall Panels

The wall panels are connected to the foundation and/or ground floor slab with dowels equal to the vertical panel reinforcing. (FEMA 178 [BSSC, 1992a], Section 8.4.6.)

G. Wood Sills

All wall elements are bolted to the foundation sill at six-foot spacing or less, with proper edge distance for concrete and wood. (FEMA 178 [BSSC, 1992a], Section 8.4.7.)

C10.5.7.2 Condition of Existing Foundations

A. Foundation Performance

The structure does not show evidence of excessive foundation movement, such as settlement or heave, that would affect its integrity or strength. (FEMA 178 [BSSC, 1992a], Section 9.1.1.)

B. Deterioration

There is no evidence that foundation elements have deteriorated due to corrosion, sulphate attack, material breakdown, or other reasons, in a manner that would affect the integrity or strength of the structure. (FEMA 178 [BSSC, 1992a], Section 9.1.2.)

C10.5.7.3 Overturning

The ratio of the effective horizontal dimension, at the foundation level of the seismic-force-resisting system, to the building height (base-to-height) exceeds $1.44A_v$. (FEMA 178 [BSSC, 1992a], Section 9.2.1.)

C10.5.7.4 Lateral Loads

A. Overturning

The ratio of the effective horizontal dimension, at the foundation level of the seismic-force-resisting system, to the building height (base-to-height) exceeds $1.44A_v$. (FEMA 178 [BSSC, 1992a], Section 9.2.1.)

B. Ties Between Foundation Elements

Foundation ties adequate for seismic forces exist where footings, piles, and piers are not restrained by beams, slabs, or competent soils or rock. (FEMA 178 [BSSC, 1992a], Section 9.2.2.)

C. Lateral Force on Deep Foundations

Piles and piers are capable of transferring the lateral forces between the structure and the soil. (FEMA 178 [BSSC, 1992a], Section 9.2.3.)

D. Pole Buildings

Pole foundations have adequate embedment. (FEMA 178 [BSSC, 1992a], Section 9.2.4.)

E. Sloping Sites

The grade difference from one side of the building to another does not exceed one-half story. (FEMA 178 [BSSC, 1992a], Section 9.2.5.)

C10.5.7.5 Geologic Site Hazards

A. Liquefaction

Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance do not exist in the foundation soils at depths within 50 feet under the building. (FEMA 178 [BSSC, 1992a], Section 9.3.1.)

B. Slope Failure

The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures, or is capable of accommodating small predicted movements without failure. (FEMA 178 [BSSC, 1992a], Section 9.3.2.)

C. Surface Fault Rupture

Surface fault rupture and surface displacement at the building site are not anticipated. (FEMA 178 [BSSC, 1992a], Section 9.3.3.)

C10.5.8 Evaluation of Materials and Conditions

C10.5.8.1 Condition of Wood

None of the wood members shows signs of decay, shrinkage, splitting, fire damage, or sagging, and none of the metal accessories is deteriorated, broken, or loose. (FEMA 178 [BSSC, 1992a], Section 3.5.1.)

C10.5.8.2 Overdriven Nails

There is no evidence of overdriven nails in the shear walls or diaphragms. (FEMA 178 [BSSC, 1992a], Section 3.5.2.)

C10.5.8.3 Condition of Steel

There is no significant visible rusting, corrosion, or other deterioration in any of the steel elements in the vertical- or lateral-force-resisting systems. (FEMA 178 [BSSC, 1992a], Section 3.5.3.)

C10.5.8.4 Condition of Concrete

A. Deterioration of Concrete

There is no visible deterioration of concrete or reinforcing steel in any of the frame elements. (FEMA 178 [BSSC, 1992a], Section 3.5.4.)

B. Post-Tensioning Anchors

There is no evidence of corrosion or spalling in the vicinity of post-tensioning or end fittings. Coil anchors have not been used. (FEMA 178 [BSSC, 1992a], Section 3.5.5.)

C. Concrete Wall Cracks

All diagonal cracks in the wall elements are 1.0 mm or less in width, are in isolated locations, and do not form an X pattern. (FEMA 178 [BSSC, 1992a], Section 3.5.6.)

D. Cracks in Boundary Columns

There are no diagonal cracks wider than 1.0 mm in concrete columns that encase the masonry infills. (FEMA 178 [BSSC, 1992a], Section 3.5.7.)

E. Precast Concrete Walls

There is no significant visible deterioration of concrete or reinforcing steel nor evidence of distress, especially at the connections. (FEMA 178 [BSSC, 1992a], Section 3.5.8.)

C10.5.8.5 Post-Tensioning Anchors

There is no evidence of corrosion or spalling in the vicinity of post-tensioning or end fittings. Coil anchors have not been used. (FEMA 178 [BSSC, 1992a], Section 3.5.5.)

C10.5.8.6 Quality of Masonry

A. Masonry Joints

The mortar cannot be easily scraped away from the joints by hand with a metal tool, and there are no significant areas of eroded mortar. (FEMA 178 [BSSC, 1992a], Section 3.5.9.)

B. Masonry Units

There is no visible deterioration of large areas of masonry units. (FEMA 178 [BSSC, 1992a], Section 3.5.10.)

C. Cracks in Infill Walls

There are no diagonal cracks in the infilled walls that extend throughout a panel or are greater than 1.0 mm wide. (FEMA 178 [BSSC, 1992a], Section 3.5.11.)

C10.6 Definitions

No commentary is provided for this section.

C10.7 Symbols

No commentary is provided for this section.

C10.8 References

ACI, 1989, *Building Code Requirements for Reinforced Concrete*, Report No. ACI 318-89, American Concrete Institute, Detroit, Michigan.

AISC, 1986, *Manual of Steel Construction—Allowable Stress Design*, Eighth Edition, Second Revision, American Institute of Steel Construction, Chicago, Illinois.

AISC, 1989, *Manual of Steel Construction—Allowable Stress Design*, Ninth Edition, Second Revision, American Institute of Steel Construction, Chicago, Illinois.

ATC, 1981, *Guidelines for the Design of Horizontal Wood Diaphragms*, Report No. ATC-7, Applied Technology Council, Redwood City, California.

ATC, 1987, *Evaluating the Seismic Resistance of Existing Buildings*, Report No. ATC-14, Applied Technology Council, Redwood City, California.

ATC, 1989, *A Handbook for Seismic Evaluation of Existing Buildings (Preliminary)*, Report No. ATC-22, Applied Technology Council, Redwood City, California.

ATC, 1995, *Structural Response Modification Factors*, Report No. ATC-19, Applied Technology Council, Redwood City, California.

ATC, 1997, *Guidelines for the Seismic Rehabilitation of Buildings: Example Application*, prepared by the Applied Technology Council for the Building Seismic Safety Council and the Federal Emergency Management Agency (Report No. FEMA 276), Washington, D.C.

BSSC, 1988, *NEHRP Recommended Provisions for the Department of Seismic Regulations for New Buildings*, 1988 Edition, prepared by the Building Seismic Safety Council for the Federal Emergency Management Agency, Washinton, D.C.

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.

ICBO, 1994a, *Uniform Code for Building Conservation*, International Conference of Building Officials, Whittier, California.

ICBO, 1994b, *Uniform Building Code*, International Conference of Building Officials, Whittier, California.

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, the Applied Technology Council, and California Universities for Research in Earthquake Engineering for the Federal Emergency Management Agency (Report No. FEMA 267), Washington, D.C.

C11. Architectural, Mechanical, and Electrical Components (Simplified and Systematic Rehabilitation)

C11.1 Scope

This chapter establishes minimum design criteria for the nonstructural components of architectural, mechanical, and electrical systems permanently installed in buildings, including supporting structures and attachments. Only a few selected contents and equipment components introduced into a building by occupants or owners are included, and these typically (though not always) would be included in the building construction documents, and as such would be subject to review by a building department.

Other equipment and contents that may be installed in the building after completion, which are not subject to building department review, are not included, even though their failure or damage may also pose significant threats to safety, building function, or property. The attempt to list all such items would result in many ambiguities and difficulties and, since they are not subject to building department review or within the typical architectural or engineering scope of services, little would be gained. The threat posed by such items must be evaluated by the engineer to the extent that the nature of such items is known through initial evaluation of the building.

In general, this chapter's component scope is similar to that of the *NEHRP Recommended Provisions* for new buildings and other model codes and standards.

C11.2 Procedural Steps

The core of this section is provided by Table 11-1, which enables the reader to establish which nonstructural components must be rehabilitated to achieve a Life Safety or Immediate Occupancy Performance Level. These requirements are also related to seismic zone. In general, the acceptance criteria are not different for the three seismic zones, but the number of types of nonstructural components that must be rehabilitated increase with the severity of the zone.

Table 11-1 also shows what kind of Analysis Method must be used for each component: a Prescriptive Procedure, a force analysis, or a combined force and relative displacement analysis. The determination of which kind of analysis is required is based on an assessment of the sensitivity of the component to acceleration or deformation, or to both. Table C11-1 shows the assumed sensitivity of the list of nonstructural components in Table 11-1 in the *Guidelines*, and which kinds of response are of primary or secondary concern.

C11.3 Historical and Component Evaluation Considerations

C11.3.1 Historical Perspective

C11.3.1.1 Background

This historical perspective presents the background for the development of building code provisions, together with a historical review of professional and construction practices related to the seismic design and construction of nonstructural components. From a historical perspective, it is important to note that mechanical engineers J. Marx Ayres and Terry Sun were among the first professionals to recognize the importance of mitigation of nonstructural hazards. After assessing building damage in Anchorage following the 1964 Alaska earthquake, they made this observation relative to building occupants:

> “If, during an earthquake, they must exit through a shower of falling light fixtures and ceilings, maneuver through shifting and toppling furniture and equipment, stumble down dark corridors and debris-laden stairs, and then be met at the street by falling glass, veneers, or facade elements, then the building cannot be described as a safe structure.” (Ayres and Sun, 1973a)

Since the 1964 Alaska earthquake, and especially since the 1971 San Fernando earthquake, the poor performance of nonstructural elements has been identified in earthquake reconnaissance reports. Subsequent editions of the *Uniform Building Code* (ICBO, 1994), as well as California and federal codes and laws have increased both the scope and strictness of nonstructural seismic provisions in an attempt to achieve better performance.

Each earthquake teaches certain special lessons concerning the vulnerability of nonstructural elements to seismic forces and displacements. Some earthquakes reveal new vulnerabilities, while most earthquakes

Table C11-1 Nonstructural Components: Response Sensitivity

COMPONENT		Sensitivity Acc.	Sensitivity Def.	COMPONENT		Sensitivity Acc.	Sensitivity Def.
A. ARCHITECTURAL				**B. MECHANICAL EQUIPMENT**			
1.	**Exterior Skin**			**1.**	**Mechanical Equipment**		
	Adhered Veneer	S	P		Boilers and Furnaces	P	
	Anchored Veneer	S	P		General Mfg. and Process Machinery	P	
	Glass Blocks	S	P		HVAC Equipment, Vibration Isolated	P	
	Prefabricated Panels	S	P		HVAC Equipment. Nonvibration Isolated	P	
	Glazing Systems	S	P				
2.	**Partitions**				HVAC Equipment, Mounted In-line with Ductwork	P	
	Heavy	S	P				
	Light	S	P	**2.**	**Storage Vessels and Water Heaters**		
3.	**Interior Veneers**				Structurally Supported Vessels (Category 1)	P	
	Stone, Including Marble	S	P				
	Ceramic Tile	S	P		Flat Bottom Vessels (Category 2)	P	
4.	**Ceilings**			**3.**	**Pressure Piping**	P	S
	a. Directly Applied to Structure	P		**4.**	**Fire Suppression Piping**	P	S
	b. Dropped, Furred, Gypsum Board	P		**5.**	**Fluid Piping, not Fire Suppression**		
	c. Suspended Lath and Plaster	S	P		Hazardous Materials	P	S
	d. Suspended Integrated Ceiling	S	P		Nonhazardous Materials	P	S
5.	**Parapets and Appendages**	P		**6.**	**Ductwork**	P	S
6.	**Canopies and Marquees**	P					
7.	**Chimneys and Stacks**	P					
8.	**Stairs**	P	S				

Acc.=Acceleration-Sensitive
Def.=Deformation-Sensitive
P = Primary Response
S = Secondary Response

present the same lessons that have yet to be learned and applied. The 1906 San Francisco, 1925 Santa Barbara, and 1933 Long Beach earthquakes pointed out the vulnerability of unreinforced brick parapets and exterior walls to seismic forces. It was obvious that—depending on the time of day and the resultant activity without and within the buildings—falling debris from the buildings might cause as great a number of casualties to pedestrians or motorists as to building occupants. It was with such potential exterior hazards in mind that the City of Los Angeles enacted a “parapet ordinance” in 1949, which required the strengthening or removal of hazardous parapets and appendages to buildings. The potential falling parapet hazard was demonstrated again during the 1952 Bakersfield, 1971 San Fernando, 1987 Whittier-Narrows, 1989 Loma Prieta, and 1994 Northridge earthquakes.

The 1952 Bakersfield, 1964 Alaska, 1983 Coalinga, and 1994 Northridge earthquakes revealed that pendant-hung and concentric ring light fixtures can fall. The 1964 Alaska earthquake first pointed out the vulnerability of modern exterior precast wall panels, elevators, and suspended ceilings. The 1971 San Fernando earthquake provided examples of the collapse of metal library shelving, debris on exit stairways, and more failures of suspended ceilings, light fixtures, and HVAC ducts. The 1989 Loma Prieta earthquake showed the dangerous collapse of some heavy plaster ceilings and ornamentation, the severe economic losses created by water damage, and the continued vulnerability of lighting grids and their supported fixtures. The 1994 Northridge earthquake produced severe problems with fire suppression sprinkler and water supply lines that failed and flooded critical hospitals, which were thus unable to perform their post-earthquake emergency response functions.

The scope of current nonstructural codes and provisions has been derived from these experiences of nonstructural failures in earthquakes, primarily in the United States, since the 1964 Alaska earthquake. Tables C11-2 and C11-3 provide a comprehensive list of nonstructural hazards that have been observed in these earthquakes.

Table C11-2 Nonstructural Architectural Component Seismic Hazards

Component	Principal Concerns
Suspended ceilings	Dropped acoustical tiles, perimeter damage, separation of runners and cross runners
Plaster ceilings	Collapse, local spalling
Cladding	Falling from building, damaged panels and connections, broken glass
Ornamentation	Damage leading to a falling hazard
Plaster and gypsum board walls	Cracking
Demountable partitions	Collapse (i.e., falling over)
Raised access floors	Collapse, separation between modules
Recessed light fixtures and HVAC diffusers	Dropping out of suspended ceilings
Unreinforced masonry walls and partitions	Parapet and wall collapse and spalling, partitions debris and falling hazard

Source: DOE, 1995

In reviewing the design and construction of architectural nonstructural components, it is useful to look at the chronological evolution of design and construction practice for these nonstructural components as part of the evolution of overall building design in this century. Focusing on office buildings as an example, four general phases can be distinguished.

Table C11-3 Mechanical And Electrical Equipment Seismic Hazards

Equipment/ Component	Principal Concerns
Boilers	Sliding, broken gas/fuel and exhaust lines, broken/bent steam and relief lines
Chillers	Sliding, overturning, loss of function, leaking refrigerant
Emergency generators	Failed vibration isolation mounts; broken fuel, signal, and power lines, loss of function, broken exhaust lines
Fire pumps	Anchorage failure, misalignment between pump and motor, broken piping
On-site water storage	Tank or vessel rupture, pipe break
Communications equipment	Sliding, overturning, or toppling leading to loss of function
Main transformers	Sliding, oil leakage, bushing failure, loss of function
Main electrical panels	Sliding or overturning, broken or damaged conduit or electrical bus
Elevators (traction)	Counterweights out of guide rails, cables out of sheaves, dislodged equipment
Other fixed equipment	Sliding or overturning, loss of function or damage to adjacent equipment
Ducts	Collapse, separation, leaking, fumes
Piping	Breaks, leaks

Source: DOE, 1995

A. Phase 1: 1900 to 1920s

Buildings featured monumental classical architecture, generally with a steel frame structure using stone facing with a backing of unreinforced masonry and concrete. Interior partitions were of unreinforced hollow clay tile or brick unit masonry, or wood partitions with wood lath and plaster. These buildings had natural (later forced-air) ventilation systems with hot water radiators, and surface or pendant mounted incandescent light fixtures.

B. Phase 2: 1930s to 1950s

Buildings were characterized by poured-in-place reinforced concrete or steel frame structures, employing columns and (in California) limited exterior and interior shear walls. Windows were large and horizontal. Interior partitions of unreinforced hollow clay tile or concrete block unit masonry, or light wood frame partitions with plaster, are gradually replaced by gypsum. Suspended ceilings and fluorescent lights arrived, generally surface-mounted or pendant. Air conditioning (cooling) was introduced and HVAC systems became more complex, with increased demands for duct space.

C. Phase 3: 1950s to 1960s

This phase saw the advent of simple rectangular metal or reinforced concrete frame structures ("International Style"), and metal and glass curtain walls with a variety of opaque claddings (porcelain enamel, ceramic tile, concrete, cement plaster). Interior partitions became primarily metal studs and gypsum board. Proprietary suspended ceilings were developed using wire-hung metal grids with infill of acoustic panels, lighting fixtures, and air diffusion units. HVAC systems increased in size, requiring large mechanical rooms and increased above-ceiling space for ducts. Sprinklers and more advanced electrical control systems were introduced, and more HVAC equipment was spring-mounted to prevent transmission of motor vibration.

D. Phase 4: 1960s to Date

Competitive battles ensued between steel and concrete frame industries. This period saw the advent of exterior precast concrete and (in the 1980s) glass fibre reinforced concrete (GFRC) cladding. Interior partition systems of metal studs and gypsum board, demountable partitions, and suspended ceiling systems become catalog proprietary items. The evolution of the late 1970s architectural style ("Post-Modern") resulted in less regular forms and much more interior and exterior decoration, much of it accomplished by nonstructural components: assemblies of glass, metal panel, GFRC, and natural stone cladding for the exteriors, and use of gypsum board for exaggerated structural concealment and form-making in interiors. Suspended ceilings and HVAC systems changed little, but the advent of office landscaping often reduced floor-to-ceiling partitions to almost nothing in general office space. After a flurry of new building forms in the late 1970s to respond to energy reduction needs—employing solar collector arrays, trombe walls, and natural ventilation systems—office building forms generally reverted to functionally or aesthetically determined configurations. In general, energy reduction is now taken care of primarily by system improvements such as insulation, lighting design, and energy-reflecting glazing. Starting in the 1980s, the advent of the "smart" office greatly increased electrical and communications needs and the use of raised floors, and increased the need for the mechanical and electrical systems to remain functional after earthquakes.

In general, seismic rehabilitation is much more likely to apply to buildings designed and constructed prior to the 1960s, with the possible exception of nonductile concrete frame buildings designed prior to the new building codes implemented in the mid-1970s. Rehabilitation may possibly apply to steel moment frame buildings found to have deficiencies in joint design or construction, but these are, for the most part, recent buildings in which nonstructural components are likely to be installed with reasonable concern for seismic performance.

C11.3.1.2 Background to Mechanical and Electrical Considerations

Prior to the 1964 Alaska earthquake, mechanical and electrical systems for buildings had been designed with little, if any, regard to stability when subjected to seismic forces. The change in design from the heavily structured and densely partitioned structures of the pre-war era, with their simple mechanical, electrical and lighting systems, to the light frame and curtain wall, gypsum board and integrated ceiling buildings of the 1950s onward, had been little reflected in the seismic building codes. The critical yet fragile nature of the new nonstructural systems was not fully realized, except for nuclear power plant design and other special-purpose and high-risk structures. Equipment supports were generally designed for gravity loads only, and attachments to the structure itself were often deliberately designed to be flexible to allow for vibration isolation or thermal expansion.

Few building codes, even in regions with a history of seismic activity, have contained provisions governing the behavior of mechanical and electrical systems until relatively recently. One of the earliest references to seismic bracing can be found in NFPA-13, *Standard for the Installation of Sprinkler Systems*. This pamphlet has been updated periodically since 1896, and seismic bracing requirements have been included since 1947. Piping systems for building sprinklers are static and do not require vibration isolation. They do, however, require flexibility where the service piping enters the building. The issue of protecting flexibly mounted piping was not studied until after the 1964 Alaska earthquake.

The designers of building mechanical systems must also address the seismic restraints required for emergency generators, fire protection pumps, and plumbing systems that are vital parts of an effective fire suppression system. The effectiveness of the requirements in NFPA-13 have been questioned based on the poor performance of some sprinkler systems in the 1989 Loma Prieta and 1994 Northridge earthquakes; opinions vary as to whether the problems lie in the requirements, in their application, or in quality control on the job. Subsequent to the Loma Prieta earthquake, the requirements were changed and augmented, but in Northridge very few buildings had sprinkler piping installed according to the 1991 NFPA standards, so the earthquake again largely tested older installations.

C11.3.1.3 Mechanical and Electrical Systems

The first systematic examination of earthquake damage to building mechanical and electrical systems occurred after the 1964 Alaska earthquake. A study by Ayres and Sun (1973a) carefully documented the damage and developed recommended corrective measures. The study was completed in 1967 but was not formally published until 1973. With the occurrence of the 1971 San Fernando earthquake, the information in this study was so important and timely that the Consulting Engineers Association of California chose to reproduce and distribute the draft report early in 1971 rather than wait for its formal publication in 1973.

Similar studies were published by the U.S. Department of Commerce following the San Fernando earthquake (Ayres and Sun, 1973b). These reports all indicated that buildings that sustained only minor structural damage became uninhabitable and hazardous to life due to failures of mechanical and electrical systems.

C11.3.1.4 HVAC Systems

The Ayres and Sun (1973b) study clearly identified the need to anchor tanks and equipment that did not require vibration isolations, and to provide lateral restraints on equipment vibration isolation devices. Some of these suggested corrective measures are now incorporated into manufactured products. The HVAC system designers had to become aware of the earthquake-induced forces on the system's components and the need for seismic restraints to limit damage; they also had to understand the requirements for the suspension and bracing of ceilings and light fixtures because of their adjacency to and interaction with the HVAC system components.

Recent significant advances in earthquake-resistive design for building mechanical systems and other nonstructural building elements have been stimulated by recurring earthquakes and the more aggressive enforcement of new building regulations, particularly by agencies such as the California Office of Statewide Health Planning and Development, and the Veterans Administration. To meet the demands of the building industry, new and improved products have been developed that assist the HVAC system designer in the preparation of construction documents. Manufacturers of vibration isolation components, hangers, supports, and restraints now offer equipment that is specifically designed to protect HVAC systems and other mechanical equipment during earthquakes. Following the 1971 San Fernando earthquake that severely damaged several hospitals, the state of California required new hospitals to be provided with the necessary seismic restraints for nonstructural components to increase the probability of hospitals remaining operational after earthquakes.

To provide technical guidance to HVAC system designers and installers, the Sheet Metal Industry Fund of Los Angeles published its first manual, *Guidelines for Seismic Restraint of Mechanical Systems* (Sheet Metal Industry Fund, 1976). This manual was updated in 1982 with assistance from the Plumbing and Piping Industry Council (PPIC) (SMACNA, 1982). The most recent manual, *Seismic Restraint Guidelines for Mechanical Equipment* (SMACNA, 1991), is designed for use in California as well as other locations with lower seismic hazard levels.

Secondary effects of earthquakes (fires, explosions, and hazardous materials releases resulting from damaged mechanical and electrical equipment) have only recently being considered. In addition, the potential danger of secondary damage from falling architectural and structural components, which could inflict major damage to adjacent equipment and render it unusable, needs to be carefully assessed.

These secondary effects can represent a considerable hazard to the building, its occupants, and its contents. Steam and hot water boilers and other pressure vessels can release fluids at hazardous temperatures. Hot water boilers operating above 212°F/100°C, in particular, represent a hazard, as the sudden decrease in pressure caused by a rupture of the vessel can result in instantaneous conversion of superheated hot water to steam, with explosive disintegration of the remainder of the vessel. Mechanical systems often include piping systems filled with flammable, toxic, or noxious substances, such as ammonia or other refrigerants. Some of the nontoxic halogen refrigerants used in air-conditioning apparatus can be converted to a poisonous gas (phosgene) upon contact with open flame. Hot parts of disintegrating boilers, such as portions of the burner and firebrick, are at high enough temperatures to ignite combustible materials with which they might come in contact (ATC, 1978).

C11.3.1.5 Building Code Provisions

The basic function of earthquake design provisions in the building code is to protect the life and safety of the public. From as early as the 1927 edition of the *Uniform Building Code* (*UBC*) until the 1961 edition, the lateral force provisions, referred to as "Lateral Bracing (Earthquake Regulations)," were only included in the *UBC* Appendix. This Appendix contained suggestions and explanatory material with reference to various details in the body of the Code, but was not considered as a legal part of the Code. Unless a locality adopted the "Lateral Bracing" provisions in the Appendix, it is reasonable to assume that there are many existing buildings that were designed without any consideration of seismic design criteria. In the 1927 *UBC*, nonstructural lateral bracing requirements were not addressed explicitly, but the Code had general wording:

> (b) Bonding and Tying. All buildings shall be firmly bonded and tied together as to their parts and each one as a whole in such manner that the structure will act as a unit. All veneer finish, cornices and ornamental details shall be bonded in the structure so as to form an integral part of it. This applies to the interior as well as the exterior of the building.

In the later editions of the *UBC*, the general wording of the 1927 *UBC* Appendix was changed to more specific horizontal force requirements for specific nonstructural components, such as nonbearing walls, partitions, curtain walls, enclosure walls, panel walls, cantilever parapet and other cantilever walls, exterior and interior ornamentation, and appendages. The first model seismic code or guideline was published in 1959 by the Seismology Committee of the Structural Engineers Association of California (SEAOC). The model codes have historically provided for lateral design of the building frame, but the evolution of provisions for nonstructural components is quite recent.

When the Lateral Bracing (Earthquake Regulations) were incorporated in the body of the *UBC* in 1961, the seismic provisions for the nonstructural components were incorporated explicitly for the first time. The horizontal lateral force that a nonstructural component and its connections were required to resist was expressed by the equation, $F_p = C_p W_p$. This equation has remained basically unchanged through the 1994 *UBC*.

Nonstructural components were referred to in the 1961 *UBC* as "parts and portions of buildings" and the scope of requirements was limited almost entirely to architectural components: nonbearing walls and partitions, masonry and concrete fences over six feet in height, cantilever parapets, and interior and exterior ornamentations and appendages. Also included were "contents, chimneys, smokestacks and penthouses, elevated tanks, and tanks resting on the ground."

There was no change in the 1964 *UBC*. In the 1967 edition "connections for exterior panels" were added, with specific requirements for these "elements" called out. There was no change in 1970. The 1973 edition added storage racks and suspended ceiling systems. In 1976 the existence of mechanical equipment was recognized by the inclusion of "rigid and rigidly mounted equipment and machinery."

Tentative Provisions for the Development of Seismic Regulations for Buildings, ATC-3-06 (ATC, 1978), presented a seismic force formula for architectural systems, mechanical and electrical components, and their attachments. This formula had five variables, including an amplification factor that increased with the height or vertical location of the component in the building. The use of such an amplification factor was not recognized in the *UBC* provisions. This amplification factor is only now fully recognized in the 1994 *NEHRP Provisions* (BSSC, 1995). Both documents include amplification factors for flexibly mounted equipment.

Some of the development in recent codes and provisions has focused on distinguishing between nonstructural components whose failure represents a life hazard, those whose failure represents primarily economic loss, and those whose failure results in loss of building function.

Particular attention has been focused on the economic consequences of nonstructural damage. The need for proper anchorage of building nonstructural elements has been clearly demonstrated by the staggering property damage and repair costs that have followed every recent earthquake. During the 1971 San Fernando, 1989 Loma Prieta, and 1994 Northridge earthquakes, code-designed buildings suffered serious damage, particularly to their nonstructural systems and components. After the 1989 Loma Prieta earthquake, the Applied Technology Council sponsored a seminar (ATC, 1992) that resulted in a series of papers that presented the latest information on the seismic design and performance of equipment and nonstructural elements. The overall conclusion of the seminar was not only to identify the problems, design deficiencies, and costs, but to restate the fact that the costs of proper restraints are minor in relation to the overall cost of the building and its contents.

C11.3.1.6 Historic Buildings

As stated in the *Guidelines*, the architectural, mechanical, and electrical components and systems of a historic building may be highly significant, especially if they are original to the building, very old, or innovative. Indeed, in many instances, both interior and exterior architectural materials and finishes may be the major argument for the preservation of the building. If this is so, than a careful assessment of their significance may be necessary by an appropriate professional such as an architectural historian, historical preservation architect, or an expert in historic material and finishes.

Sometimes removal of later finishes may reveal materials or finishes of historic value in a building not specifically identified as historic. Again, careful assessment by a qualified expert is necessary.

A careful nonstructural mitigation plan is necessary to ensure that historic materials and finishes are preserved, while still meeting the requirements for the specified Rehabilitation Objective.

While the architectural materials and finishes in historic buildings are commonly of major historic interest, it is also possible that mechanical or electrical components, or plumbing fixtures, will be of historic value and should be preserved. On the other hand, historic buildings may also have materials—usually concealed, such as lead pipes or asbestos—that may pose a hazard, depending on their location, condition, use or abandonment, and/or disturbance during the rehabilitation. Such problems must also be identified as part of the rehabilitation plan, and steps taken to ensure requisite safety for workers and occupants.

C11.3.2 Component Evaluation

A suggested general procedure for developing a mitigation plan for the rehabilitation of nonstructural components is as follows.

1. It is assumed that the building has been evaluated in a feasibility phase, using a procedure such as that described in FEMA 178 (BSSC, 1992b). For nonstructural components, use of this procedure will have provided a broad list of deficiencies generally, but not specifically, related to a Rehabilitation Objective.

   Issues related to other objectives and possible nonstructural components not discussed in FEMA 178 (BSSC, 1992b), as well as issues raised by nonstructural rehabilitation unaccompanied by structural rehabilitation (e.g., planning, cost-benefit) are outlined in this *Commentary*, and references are provided for more detailed investigation.

2. The decision is made to rehabilitate the building, either structurally, nonstructurally, or both.

3. From Chapter 2 in the *Guidelines*, the designer reviews Rehabilitation Objectives and, in concert with the owner, determines the Objective; alternatively, the Objective may already have been defined in an ordinance or other policy.

4. Armed with a decision on the Rehabilitation Objective, which includes Performance Level or Range as well as ground motion criteria, the designer consults Chapter 11 of the *Guidelines*.

5. Using Chapter 11, the designer prepares a definitive list of nonstructural components that are within the scope of the rehabilitation, based on the selected Performance Level and an assessment of component condition. For the Life Safety Level and, to some extent, the Immediate Occupancy Level, Chapters 2 and 11 in the *Guidelines* specify requirements. However, for other levels and ranges, there is a need to evaluate and prioritize. A suggested procedure is outlined in Sections C11.3.2.1 through C11.3.2.9.

6. From the list of nonstructural components within the project scope, a design assessment is made to

determine if the component requires rehabilitation and, from Table 11-1 in the *Guidelines*, the rehabilitation Analysis Method (Analytical or Prescriptive) for each component or component group is determined.

7. For those components that do not meet the criteria, an appropriate analysis and design procedure is undertaken, with the aim of bringing the component into compliance with the criteria appropriate to the Performance Level or Range and the ground motion criteria.

8. Nonstructural rehabilitation design documents are prepared.

C11.3.2.1 Overview

The nonstructural evaluation procedure set out in this section can be used for the development of a mitigation plan incorporating priorities related to achieving a selected Rehabilitation Objective (or Objectives) within available resources.

A formal evaluation procedure is suggested in order to establish the real relative risks posed by the nonstructural components. While Table 11-1 in the *Guidelines* identifies the relationship between nonstructural components, seismic zones, and the rehabilitation requirement and analysis procedures to meet the Life Safety and Immediate Occupancy Performance Levels, it is necessary to prepare a definitive list of nonstructural components for rehabilitation, based on assessment of risk, priority, and available budget.

A suggested nonstructural evaluation procedure is summarized in Figure C11-1. The procedure includes the following steps:

1. A preliminary evaluation based on FEMA 178 (BSSC, 1992b)

2. Selection of a desired Rehabilitation Objective for the building

3. A building "walk-down" to establish an inventory of nonstructural components that includes:

   a. Locations and quantities of selected components, and vulnerabilities and consequences of failure of each component

   b. Development of a seismic risk rating for each component

4. Development of a mitigation priorities list

5. Establishment of Analysis Method from Table 11-1

6. Development of appropriate rehabilitation design concepts

7. Preparation of a performance-related mitigation plan

A final mitigation plan, developed in concert with the owner, must also relate costs to available budget and possible time constraints. When these factors are considered, the selected Rehabilitation Objective may have to be modified, planned to be accomplished in a phased program, or both. These additional steps are shown in Figure C11-1.

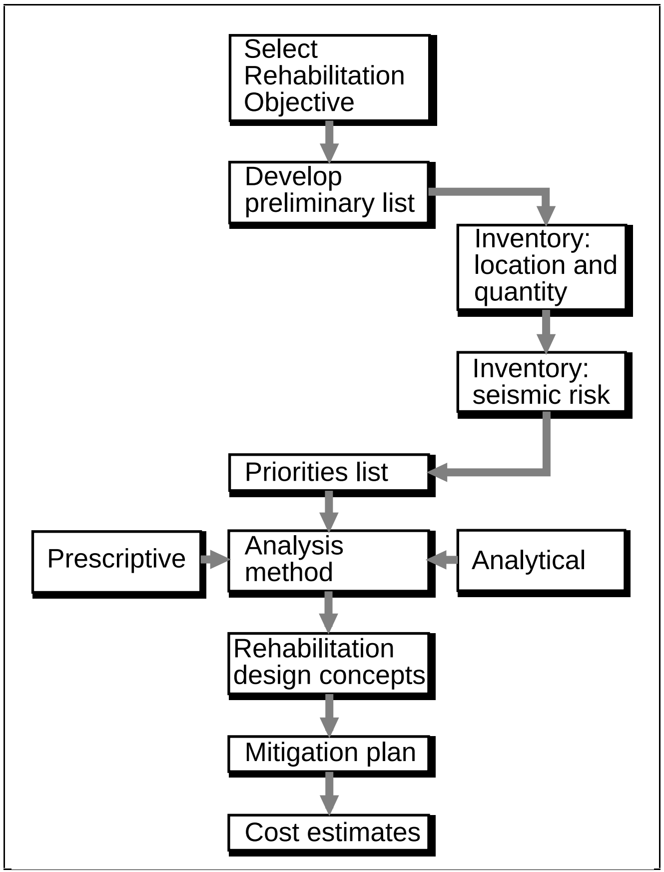


Figure C11-1 Nonstructural Evaluation Procedure

C11.3.2.2 Preliminary Evaluation

The *NEHRP Handbook for the Seismic Evaluation of Existing Buildings*, Chapter 10, "Evaluation of Elements that Are Not Part of the Lateral-Force-Resisting System" (FEMA 178) (BSSC, 1992b), provides the basic criteria for the evaluation of nonstructural elements. If a FEMA 178 nonstructural evaluation has been performed on the building, a copy should be obtained and its findings evaluated as a preliminary to using the "walk-down" procedures discussed in Section C11.3.2.4.

It is important to note that the FEMA 178 (BSSC, 1992b) evaluation statements and performance characteristics are all-inclusive, and do not differentiate between nonstructural elements that are specifically life-safety hazards and those elements whose seismic performance relates more to Damage Control and Immediate Occupancy goals.

For buildings with Life Safety Performance Level goals, no further evaluation work need be undertaken for systems for which the FEMA 178 (BSSC, 1992b) evaluation statements can all be answered "True," resulting in a "Low" vulnerability rating. When evaluation statements receive "False" answers, additional investigation needs to be undertaken in accordance with the following procedure.

For buildings with Enhanced Rehabilitation Objectives, the vulnerability assessment described above must be augmented with an assessment of seismic risk that considers property loss and loss of building function. This is done by use of information in FEMA 74 (FEMA, 1994) as described in Section C11.3.2.4.

C11.3.2.3 Rehabilitation Objectives

One or more Rehabilitation Objectives must be selected, prior to further evaluation of in-place conditions or analysis of rehabilitation measures.

C11.3.2.4 Building Walk-Down: Inventory, Location, Quantity, and Seismic Risk

In order to assess the extent of the real nonstructural problems in an existing building that is under evaluation for seismic rehabilitation, a formal "diagnosis" is necessary. This ensures that all items are accounted for, and that a reasonably standardized procedure is followed that will result in a balanced assessment of risk, cost, and priority.

One effective diagnostic measure is the seismic survey or "walk-down" inspection. The walk-down inspection process begins by developing an inventory of important architectural components and mechanical and electrical equipment. The list of components in Table 11-1 of the *Guidelines* provides the basis for this, but items may be added or subtracted depending on the Rehabilitation Objective and the nature of the specific building.

The nonstructural seismic "walk-down" has two main objectives:

1. To inventory the nonstructural items that are considered important, and to establish their location and quantity
2. To establish for each component, item, or system, its seismic risk, which is a combination of seismic vulnerability and the consequences in relation to the seismic Rehabilitation Objectives

Appendix A of FEMA 74 (FEMA, 1994) provides a suitable inventory form, together with an example of how it is used. Teams involved in the development of inventories may wish to design forms appropriate to their office practice, the nature of the project, and the level of detail that the owner requires.

Not all data need be collected in every instance. For Limited Rehabilitation Objectives—or in situations where rehabilitation does not depend on particular information, such as quantity—only sample data are necessary.

The seismic risk assessment of each item is best accomplished by a two-person team of architects and/or engineers experienced in seismic design and evaluation of the seismic performance of the building's structural and nonstructural elements. The Checklist of Nonstructural Earthquake Hazards in Appendix B of FEMA 74 (FEMA, 1994), and the Nonstructural Risk Ratings of Appendix C, can be used to assess whether the nonstructural components present a danger to building occupants (in cases where their proximity to occupied space is critical) or are likely to cause financial loss or operational interruption following an earthquake.

For more guidance on the assessment of nonstructural risk rating, refer to the beginning of Appendices B and C in FEMA 74.

C11.3.2.5 Priority Setting

If a Rehabilitation Objective other than the BSO, or voluntary rehabilitation with objectives defined by the owner, is being pursued, it will be necessary to establish priorities for the rehabilitation of nonstructural components. To do this, some of the items in the inventory (determined by the building walk-down) may need a further level of evaluation. The level of formality in this evaluation may vary from some discussion among the principal participants for a small project, to preparation of a carefully prepared list for a large project. The setting of priorities is of particular importance in a large project for which the budget for nonstructural rehabilitation is limited.

In the preparation of a careful prioritized list that can form the basis for budgetary discussion, the information derived from the use of the two checklists in Appendices B and C of FEMA 74 (FEMA, 1994) to establish a risk rating for each component can be further refined by recognizing that seismic risk is a combination of "vulnerability" and "consequences."

"Vulnerability" is an estimate of the likelihood of component failure; it is assessed as a measure of:

1. The characteristics of the ground motion
2. The response of the building in terms of acceleration and displacement
3. The size and weight of the element
4. Its location in the building (e.g., the first floor or roof)
5. The type of building lateral-force-resisting system and the relative stiffness of the structure and the nonstructural element
6. The adequacy of the connection or lack of connection of the nonstructural component to the structure and other supporting nonstructural elements

"Consequences" is an estimate of the effect of component failure; it relates to:

1. The item's location in the building
2. The building occupancy and function, and the potential impact on life safety and/or building function if the component or equipment were to fail

In addition, some components, such as appendages and cladding, must be evaluated in relation to adjacent—and possibly lower—buildings, alleys, parking areas, sidewalks, plazas, parks, and landscaped areas.

Typically, the assessments are made on the basis of visual observation and engineering judgment, either during the building walk-down, or as a separate activity after it is conducted. For the most part, no formal seismic calculations are performed or reviewed in these assessments. However, when faced with items of high consequence and questionable seismic resistance, it may be necessary to do a structural analysis using the default equation (Equation 11-1) in the *Guidelines*. This is the only reasonably sure way to establish that a particular element has the desired level of seismic resistance, particularly in the high seismic areas of the United States.

The Seismic Vulnerability ratings are as follows:

Low Seismic Vulnerability: The identified component is reasonably well anchored, and there is a low probability of it failing under the design forces and deformations of the building.

Moderate Seismic Vulnerability: The identified component is anchored, but there is a moderate probability of it failing under the design forces and deformations of the building.

High Seismic Vulnerability: The identified component is either poorly anchored or not anchored, and there is a high probability of it failing under the design forces and deformations of the building.

The Seismic Consequence of Failure ratings are as follows:

Low Seismic Consequence: The identified component is so located in the building or is of such a type that its failure represents a low risk (no injury or minor injury) to the occupants and a low adverse impact on the seismic Performance Level for the building.

Moderate Seismic Consequence: The identified component is so located in the building or is of such a type that its failure represents a moderate risk (minor to moderate injury) to the occupants and a moderate adverse impact on the seismic Performance Level of the building.

High Seismic Consequence: The identified component is so located in the building or is of such a type that its failure represents a high risk (death or serious injury) to the occupants and a high adverse impact on the seismic Performance Level of the building.

In a nonstructural seismic rehabilitation project, the obvious nonstructural risks to be rehabilitated first would be those hazards that have a high probability of causing injury and/or death to the occupants, or to those people entering, leaving, or adjacent to the building. These hazards would have High Seismic Consequence ratings. These High Seismic Consequence nonstructural hazards should then be further ranked for rehabilitation according to their High, Moderate, and Low Seismic Vulnerability ratings. To assist in the evaluation, the ratings of Vulnerability and Consequences for components whose priority is not clear can be tabulated as shown in Table C11-4.

Table C11-4 Nonstructural Rehabilitation Priority Ratings

Vulnerability Rating	**Consequence Rating**		
	High	Moderate	Low
High	1	4	7
Moderate	2	5	8
Low	3	6	9

Given the combined Seismic Vulnerability and Consequence rating, the order in which the nonstructural hazards should be rehabilitated is provided by the rank order of the number in Table C11-4: 1 is the highest priority, 2 is the second, 3 is the third, and so on.

The priority setting of the seismic rehabilitation of the nonstructural element is primarily governed by the level of the Seismic Consequence rating, and second by the Seismic Vulnerability rating. Since the determination of the consequences of a nonstructural element failing can generally be made with a higher degree of certainty than its seismic vulnerability—because the evaluation criteria earthquakes could be exceeded—the seismic consequence rating is the key predictor variable.

A nonstructural element with a Low Seismic Consequence rating would not have a high priority for rehabilitation regardless of its Seismic Vulnerability rating.

An example would be a heavy concrete exterior cladding panel, improperly attached to the structure, which would have a High Seismic Vulnerability rating. However, if this cladding panel were located above a light well where occupant and public access were restricted, it would have a Low Seismic Consequence rating. As long as the restriction were maintained, this cladding panel would have low priority, a ranking of 7, for rehabilitation, with a Limited Safety Performance Level goal for the building. If the Performance Level goal for the building was Immediate Occupancy and the local climatic conditions were such that proper enclosure of the building from the weather was necessary, then the seismic rehabilitation of the inadequately anchored heavy panel would probably have a high priority.

In buildings with Life Safety Performance Level goals, the potential falling hazard of an improperly anchored heavy light fixture in an exit corridor, with a High Seismic Vulnerability rating, and a High Seismic Consequence rating, should have a higher priority for rehabilitation than a similar light fixture in an infrequently occupied storage area with a lower Seismic Consequence Rating. The same argument can be made that improperly installed lay-in T-bar ceiling systems in exit corridors should have a higher rehabilitation priority than similar ceiling systems over office work areas.

In buildings for which the Damage Control Performance Range or Immediate Occupancy Performance Level is a goal, it would be necessary to rehabilitate all nonstructural hazards throughout the building—regardless of the Consequence Rating—starting with the rehabilitation of the High Seismic Vulnerability rated elements, to reduce the vulnerability to less than Low.

Many other patterns of priority—based on specific Rehabilitation Objectives, building conditions, resources, and site seismicity—can be envisaged.

C11.3.2.6 Analysis

For those components requiring rehabilitation, an analysis should be undertaken, based on the procedures described in Section 11.7.3 or 11.7.4 and in sections relating to specific components.

C11.3.2.7 Rehabilitation Concept Development

Based on the rehabilitation procedure, a design concept can be assigned and quantified.

C11.3.2.8 Cost Estimating

A cost estimate should be prepared for each identified component and priority ranking.

C11.3.2.9 Nonstructural Component Hazard Mitigation Plan

Based on the evaluation, priorities, rehabilitation procedure, costs, and available resources, a mitigation plan should be prepared that establishes the objectives, rehabilitation type, order, estimated cost, and suggested time frame for nonstructural hazard mitigation.

C11.4 Rehabilitation Objectives, Performance Levels, and Performance Ranges

A Rehabilitation Objective combines ground motion criteria (mean return period of earthquake related to standardized maps)—which is stated in the *Guidelines* in terms of probabilities in 50-year exposure periods—with a description of acceptable behavior of the building (Performance Level or Performance Range). The Basic Safety Objective (BSO) defined in the *Guidelines* includes both structural and nonstructural requirements, because one of the two Performance Levels required for that Objective to be met is Life Safety. The BSO is a basic benchmark, and thus its inclusion of nonstructural requirements is a significant part of the *Guidelines*.

The two ground motion analyses required in the BSO, BSE (Basic Safety Earthquake)-1 and BSE-2, are applied to the Life Safety and Collapse Prevention Performance Levels, respectively. (See Chapter 2.) However, Collapse Prevention criteria relate—with one exception—only to the building structure, although nonstructural components that modify the structural response (such as nonstructural infill walls) must also be considered. The exception is that parapets and appendages should also be rehabilitated at the Collapse Prevention Performance Level, because the result of their failure—massive falling debris—is analogous to that of the structure. It would not make sense to rehabilitate a structure without also dealing with parapet and appendage problems.

Typically, the Rehabilitation Objective for nonstructural components will be the same as for the building structure. However, an owner might choose to rehabilitate nonstructural components to a higher level in a given project, for purposes of damage control (to reduce economic losses). In another case, a structure might be adequate to meet the Life Safety Performance Level or even Immediate Occupancy, but because nonstructural rehabilitation would be very costly to achieve for those levels, the owner may choose not to attempt it.

It is also possible for nonstructural rehabilitation to be provided in the absence of any structural rehabilitation; for example, where the structure is already found acceptable, or where seismic risk is relatively low and structural performance is likely to be good. In these cases, nonstructural rehabilitation may be justified, because nonstructural damage can occur at relatively low accelerations in minor to moderate events, and reducing nonstructural damage can be very cost-effective compared to the costs of damage and business interruption if nonstructural components are left unrehabilitated.

C11.4.1 Performance Levels for Nonstructural Components

When the BSO is selected, all nonstructural components that are identified in Table 11-1 of the *Guidelines* as relevant to the Life Safety Performance Level must meet specific requirements for the BSE-1 ground motion. In some cases, judgment must be used to determine the life safety implications of certain nonstructural components for a specific building, such as the evaluation of pendant light fixtures to determine their hazard potential.

While some items—such as much mechanical equipment—pose a very low life-safety threat, and hence rehabilitation is (with some exceptions) generally not required, owners might be wise to rehabilitate these items because the techniques are simple and inexpensive, and the benefits in reduction of property loss are great.

Criteria for nonstructural components for more severe ground motion, or for the Immediate Occupancy or

Operational Performance Levels, provide for Enhanced Rehabilitation Objectives that meet and exceed the BSO.

Table 11-1 in the *Guidelines* establishes the list of nonstructural components included within the scope of the detailed requirements of the *Guidelines*. Where the Life Safety Performance Level is applicable, the components indicated in Table 11-1 as Life Safety components must meet the specific acceptance criteria given in Chapter 11 of the *Guidelines*. For individual components in Sections 11.9, 11.10, and 11.11, acceptance criteria are also provided that relate to the Immediate Occupancy Performance Level.

On a single project, Nonstructural Performance Levels may be combined. The criteria for parapets would often be those of the Life Safety Performance Level. In the same building, art objects, telephone and computer room components, or the backup motor-generator set and its associated cooling, fuel, and other components, might rationally be protected up to the Operational Performance Level. In this same building, there may be some nonstructural features listed in Table 11-1 whose rehabilitation is deferred; for example, adhered veneer in a low occupancy area might be difficult to rehabilitate, and a decision might be made to not include it within the project. Thus, unlike structural components, the nonstructural components in a single building have often been assigned a mixture of Performance Levels in the rehabilitation process, and this flexibility has been maintained in the *Guidelines*. As defined in Chapter 2, an overall Building Performance Level is the combination of one Structural Performance Level or Range and one Nonstructural Performance Level or Range. To satisfy the Life Safety Performance Level, all of the nonstructural requirements of the Nonstructural Life Safety Performance Level must be met.

It is recognized that the failure of an architectural, mechanical, or electrical component might have an adverse effect on code-required life safety systems, but the intent of the *Guidelines* for the Life Safety Performance Level is limited to ensuring that all architectural, mechanical, and electrical systems remain intact to the extent that they do not create a falling hazard, an ignition hazard, or release of materials that are hazardous for short-term exposure.

Rehabilitation to an Operational Performance Level implies a damage state in which the building is immediately suitable for occupancy and use, albeit in a somewhat impaired mode; acceptable impairments will vary depending on the building occupancy.

The Operational Performance Level represents a level above Immediate Occupancy; the focus is on maintaining utility services within the building together with essential equipment that would vary according to the building function. The structural state might be identical with Immediate Occupancy.

No specific criteria for nonstructural components for the Operational Performance Level are 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. For example, the continued operation of a hospital emergency room may depend on sophisticated medical equipment, which has not been designed with the seismic problem in mind. While use of the *Guidelines* may ensure that such equipment is adequately braced or anchored, the design team cannot evaluate the resistance capacity of a closed piece of equipment—a so-called “black box”.

Depending on the importance of the equipment and the resources available to the design team, seismic testing and certification of such equipment may be requested of the manufacturer. Alternatively, special attention may be paid in the rehabilitation design to reducing the building response, and hence the likelihood of equipment failure, by use of advanced design techniques such as base isolation and energy dissipation.

Experience in recent earthquakes—notably, the 1994 Northridge event—has revealed the difficulties inherent in the attempt to ensure post-earthquake operational capacity. After the Northridge earthquake, some major new hospitals with current-practice nonstructural seismic features for essential facilities had to shut down due to nonstructural damage, because their nonstructural features were both unusually complicated and very essential. Even where good seismic detailing prevented a large amount of damage, a few seemingly small failures (for example, one or two pipe breaks) were sometimes enough to cause large disruptions in some buildings (Hall, ed., 1995). Clearly, in a complex building with thousands of feet of piping and hundreds of joints and connections, it is very hard to provide a zero-defect system.

Water leakage may have serious interactive effects, affecting the operation of an otherwise functional backup power system. At Northridge, the power outage was so extensive—affecting two million customers throughout Los Angeles and some other communities—that reliable backup power was necessary for essential facilities to operate. Even in some buildings with extensive backup power systems that functioned correctly after the earthquake (such as at the Veterans Administration Sepulveda Medical Center), water leakage caused short circuits and power was automatically shut off. At Holy Cross Hospital, one patient on life support died because the properly functioning backup system stopped when sprinkler pipe leakage caused wiring to ground out.

Experience has also shown that both approaches to the overall design of a system (besides correct detailing and installation) and managerial responses may play an important role in ensuring operational capability. Based on disruptive sprinkler and other piping leakage in the Northridge earthquake, some suggestions have been made for essential facilities (in addition to trying to prevent leakage). These are (1) zoning systems into smaller areas, so that smaller areas can be shut off; (2) providing automatic or remotely controlled valves; and (3) more rigorously training designated personnel in shut-off techniques. Even with damage to critical data processing equipment, the overall impact on facility function can be minimized with a redundant or backup site and rapid response (Holmes and Reitherman, 1994).

The lesson of the Northridge earthquake appears to be that good seismic detailing and careful installation to meet the acceptance criteria for Life Safety and Immediate Occupancy will go a long way in protecting essential equipment and services, but that complex facilities remain vulnerable to even a single failure in a complex system. If reliability of systems is critical, careful building-specific evaluation and design are necessary, and techniques such as base isolation and specially designed and redundant systems, as in nuclear power plants, must be considered.

C11.4.2 Performance Ranges for Nonstructural Components

Nonstructural rehabilitation within a Limited Safety Performance Range below the BSO might include mitigation of the hazards of some, but not all, of the nonstructural components identified as Life Safety components in Table 11-1, or for elements considered hazardous at a more probable (less intense) level of shaking than the BSE-1 criterion. Rehabilitation techniques should be designed for criteria that meet or exceed BSE-1 wherever feasible. This is not likely to incur a cost or design complexity penalty.

Included within a variety of partial rehabilitation measures is the Nonstructural Hazards Reduced Performance Level. There are numerous actual examples of nonstructural seismic rehabilitation of this type. For example, if a remodelling project afforded the inexpensive opportunity to rehabilitate the ceilings, partitions, and other components on the Life Safety list of Table 11-1, but the components in another portion of the building were not included, the project would not fully meet the Life Safety Performance Level. Alternatively, throughout a building only some of the components on the Life Safety list of Table 11-1 might be rehabilitated; for example, by some cost-benefit decision-making process, the heavy light fixtures might be restrained from falling, while the more expensive bracing of the lightweight ceiling might not be rehabilitated. This level of rehabilitation would be the Hazards Reduced Performance Level. Except for the Hazards Reduced Performance Level, the *Guidelines* do not define a particular set of components that must be rehabilitated to meet the requirements of these ranges. The *Guidelines* also do not specify which kind of risk-reduction goal—prevention of injury, protection of property, or provision for continued post-earthquake operation—must be considered for these ranges.

Nonstructural rehabilitation exceeding the Life Safety Performance Level might include post-earthquake functionality protection for nonstructural components or features such as emergency escape and rescue routes, data processing or communications equipment and services, or other activities that are occupancy-related. Protection of property—such as protecting brittle architectural features of a building from cracking even if the cracking would not be hazardous—is another example. In addition, rehabilitation within this range might focus extensively on contents, such as valuable art artifacts, that are not within the scope of the *Guidelines*.

In general, once the Life Safety Performance Level requirements are met, a significant degree of protection from functional failure and property damage is also achieved, but this varies greatly from building to building and may not approach the specific occupancy-related expectations of post-earthquake functionality.

C11.4.3 Regional Seismicity and Nonstructural Components

No commentary is provided for this section.

C11.4.4 Means of Egress: Escape and Rescue

C11.4.4.1 Background

The ability of building occupants to safely leave a building immediately after an earthquake, or for response personnel to enter it for rescue purposes, is a recognized seismic rehabilitation issue. To achieve these ends, the intent of some rehabilitation has included keeping the "means of egress" moderately free from obstruction after the earthquake. In the development of this document, an attempt has been made specifically to define this subject area, and in the process, it was found that the scope of this issue is much broader than is often assumed. As a result, the option of embedding egress criteria, or emergency escape and rescue requirements, in the *Guidelines* for the Life Safety Performance Level was rejected, primarily for two reasons.

1. Criteria for means of egress and exiting have many code implications beyond those generally thought to be relevant for the post-earthquake situation. If this document required that means of egress be provided for the post-earthquake Life Safety Performance Level, this might also trigger many code requirements not specifically related to post-earthquake safety, which would be difficult and costly to implement.

2. Previous documents' references to egress were felt broadly to imply a guarantee that virtually all circulation routes and related nonstructural features and services required by current code would be functional in the post-earthquake setting. That expectation most closely matches the definitions of the Immediate Occupancy Performance Level, or in some cases the Damage Control Performance Range, rather than the Life Safety Performance Level.

The following discussion explains the background to this issue, and offers some guidance on how to effectively include provision for this concern in designing for an Immediate Occupancy Performance Level or within a Damage Control Performance Range.

C11.4.4.2 Code Implications of Means of Egress

The term "means of egress" has a particular meaning in model building codes: that of the provision for *exits*, which include "intervening aisles, doors, doorways, gates, corridors, exterior exit balconies, ramps, stairways, pressurized enclosures, horizontal exits, exit passageways, exit courts and yards" (*Uniform Building Code* [ICBO, 1994], Chapter 10, Definitions). Other model codes use essentially the same definitions, because historically egress requirements were included because of fire hazard. Very specific criteria are provided for all the above items.

The *UBC* does not distinguish between egress (exiting) and ingress (entering), and the latter term does not appear at all in the *UBC*. For the post-earthquake situation, egress is the governing condition: if egress is preserved for the occupants, then rescue personnel—who will almost certainly have equipment (e.g., lights and tools) that the occupants may lack—should have little difficulty entering the building. In the following discussion, the term "egress" refers to both egress and ingress.

Egress differs significantly from "access" in code terminology. Access is literally the ability to approach and go into the building (that is, the same as ingress), but access and accessibility now have a specific code definition as "complies with this chapter and that can be approached, entered and used by persons with physical disabilities" (*UBC*, Chapter 11, Accessibility, Section 1102, Definitions).

In building code use, disabled accessibility refers to two-way (ingress and egress) capability: disabled people are supposed to be able to go into the building, and there are also special requirements to aid their egress, which together are called "accessibility provisions."

The imposition of requirements aimed at the post-earthquake protection of "means of egress" without qualification can thus complicate post-earthquake escape and rescue needs by triggering a long list of nonseismic code requirements. Triggering of building code requirements to upgrade exits might, for example, include the widening of corridors, addition of stair towers, or installation of wheelchair-accessible ramps.

Such nonseismic issues are not embedded in the requirements of the *Guidelines*.

To include a phrase such as "maintain all exits and exitways" in the *Guidelines* could also be construed to require installation of a complete emergency power system where none would otherwise be required, because exit signs, stairwell lights, annunciation systems, and other electrically powered components required by code for exiting concerns might not operate after an earthquake, when experience has shown that widespread power outages must be regarded as routine.

The ability to enter and circulate safely through a building in continuation of its normal operation, which is part of a building code's intent, is not a seismic life safety-related concern and thus is distinct from the subject discussed here. Therefore, ensuring that escalators continue to function in a department store is a concern related to Immediate Occupancy and protection of business operations rather than to Life Safety, and seismic rehabilitation to protect the ability of escalators to function would be based on criteria considerably more restrictive than simply emergency post-earthquake escape and rescue.

C11.4.4.3 Life Safety Performance Level and Post-Earthquake Conditions

The Life Safety Performance Level is directed toward the limited objective of reducing, to a low but unspecified probability, casualties caused by structural or nonstructural damage. As a practical matter, the injury-prevention aim in most cases would impose more restrictive requirements on nonstructural components than any specific criteria for preventing obstruction to means of egress. The Life Safety Performance Level requires that the most hazardous nonstructural components are replaced or rehabilitated. As stated in the *Guidelines*, the items listed in Table 11-1 for achieving the Life Safety Performance Level show that typical requirements for maintaining egress—such as the items listed in the *NEHRP Handbook for the Seismic Evaluation of Existing Buildings*, pp. 91–92, and p. A-20 (BSSC, 1992b), which must apply if that document's definition of life safety is to be met—would be taken care of. These items are listed in the *Guidelines*, as well as five other potential obstructive hazards.

Beyond those provisions for architectural nonstructural components, the requirements for preserving the means of egress become very broad, and specific to the building and occupancy type. Some examples follow.

Provision of emergency power may be a wise investment; it has been required by ordinance in some communities for nonseismic safety concerns, as in the common case of battery-powered flood lamp units added to stairwells or over some exit doors in programs enforced by local fire departments.

One can argue that provision of emergency lighting could improve post-earthquake escape and rescue movement through a building as much as—or more than—prevention of the falling of some of the suspended acoustic ceiling. Various cost-benefit evaluations are possible; the results will depend on the specifics of the building and its occupancy.

Security and fire alarm systems have sometimes been falsely set off by power fluctuations caused by earthquakes; direct damage to these components, or a power outage in the absence of backup power, can also cause an outage, and adversely affect escape and rescue.

In a high-rise building, specific annunciator system requirements are stipulated by (nonseismic) building codes, and it can be argued that functionality of these systems is important for escape and rescue in the post-earthquake situation if the building catches fire. However, to insist on complete rehabilitation or replacement of building security systems as a Life Safety Performance Level item would be an expensive measure for a very low-probability event.

Similarly, building and fire codes contain numerous requirements related to fire and hazardous material safety, including provision of smoke-free shafts, hazardous material exhaust systems, and a backup supply of water for sprinkler systems.

The fire rating of a door assembly or wall can be affected by racking and seemingly minor cracking; thus, if a seismic performance definition requires a building to maintain full "fire safety," this could imply that virtually no damage is to occur. Part of the rationale for limiting post-earthquake building egress requirements is based on the low probability that the earthquake would cause a fire or hazardous material release that would pose an immediate threat to occupants, so long as they were able to leave within a reasonable amount of time.

The *Guidelines* have carefully kept the evaluation and rehabilitation of components and systems such as the above, and others, as options separate from the basic definition of the Life Safety Performance Level, to preserve the clear meaning of the Level's intent: prevention of earthquake damage that can directly injure people.

C11.4.4.4 Issues of Maintaining Post-Earthquake Means of Egress

If the comprehensive set of building egress concerns (e.g., lighting, elevators, alarms) are selected as part of the Damage Control Range or Immediate Occupancy Level, then much more extensive rehabilitation measures would be required. Some of the major areas of concern are discussed below.

A. Critical Escape and Rescue Areas

This term has no preestablished definition, but the intended meaning is that of a hallway, stairwell, or fire escape, an entry space such as a lobby, or an exterior area outside an exit doorway. The intent is that such areas might be especially deserving of additional nonstructural seismic protection.

Occupant loads passing through a doorway that is required as part of an exit pathway can be calculated according to building codes and standards, and any doorway with a load over some particular amount might also be defined as critical. Redundancy of pathways would also be logically involved in determining which areas are most critical and deserving of nonstructural protection.

On a smaller scale, localized areas in rooms are more critical for access than others. For example, tall bookshelves and cabinets located next to an inward-opening door have toppled, blocking access to the room. After the 1989 Loma Prieta earthquake, it took approximately an hour for a rescue crew to obtain access to the Watsonville Community Hospital cafeteria, for just this reason.

Thus, to determine a rehabilitation strategy as to which circulation areas are more critical than others would require careful study of building and occupancy specifics, and coordination with locally applicable retroactive fire safety standards, to derive an appropriate design strategy.

B. Occupancy

Building codes have traditionally defined types of occupancies for purposes of setting fire safety provisions, and dozens of building and fire code requirements are keyed according to very specific occupancy classes and sub-classes, with rules for calculating numbers of occupants. Building egress provisions are then related to these occupant classes and loads. For purposes of post-earthquake escape and rescue, some of the code-determined occupancy classes and/or loads can be used for assessing the importance of critical escape and rescue areas.

C. Obstructions

Major obstruction could be defined as debris or damage that makes escape or rescue more difficult than climbing through a code-minimum rescue and escape window, which is required in U.S. building codes for sleeping rooms from the basement through the third story.

This requirement for escape windows is aimed primarily at fire: the small dimensions that the code regards as acceptable for safety (minimum height 24 inches/610 mm and minimum width 20 inches/508 mm) should be noted. For the post-earthquake situation, most occupants would probably expect a less limiting criterion.

D. Elevators

Rehabilitation of elevators is aimed at safety rather than immediate operation, and their use for immediate escape is not contemplated. Current seismic provisions for elevators are aimed at safe shutdown, rather than continued functionality. After even moderate shaking, any elevator will need inspection after shutdown before it can be regarded as safe. This fact alone means that elevators cannot be regarded as available for escape or rescue.

People in wheelchairs cannot be easily carried down stairs, so when elevator service is disrupted by an earthquake (either because of lack of backup power, disruption of a backup system, automatic shutoff without rapid inspection and restoration, or direct damage to the elevator system), the egress capability of people with movement disabilities is further impaired.

E. Sprinkler Systems

It is sometimes argued that, because of the possibility of post-earthquake fires, protection of sprinkler systems should be part of the Life Safety Performance Level requirements. However, fires in buildings are a relatively low-frequency occurrence. Moreover, fires take some time to develop, so the threat to life is minimal in the first minutes after an earthquake if the means of egress are reasonably intact. However, the bracing of sprinkler systems is a major property protection issue, and since there is a life safety issue involved, rehabilitation of sprinkler systems is required in the *Guidelines* as part of the Life Safety Performance Level acceptance criteria in high seismic areas. Again, the Life Safety Level in the *Guidelines* is limited to the threat of direct injury.

F. Water Leakage

From the standpoint of escape and rescue, minor water leakage can be considered more of a nuisance than a life safety issue, but there are cases where leaking water can make the use of stairs or other exit routes as difficult as if there were debris in the way, and there may be electrical shock concerns as well. A building-specific evaluation would be necessary to determine the likelihood of such consequences in critical escape and rescue areas of the building. Water leakage has been proven to be a major source of economic loss, and a cause of building operational loss in essential buildings such as hospitals.

While a strict adherence to the requirements for Life Safety may reduce the cost and extent of nonstructural rehabilitation, the prudent owner may in fact find that the twin objectives of safety and reduced property loss are best served by a building-specific program that encompasses a wide range of the requirements for improving the means of egress in the post-earthquake situation.

C11.5 Structural-Nonstructural Interaction

C11.5.1 Response Modification

When the nonstructural component affects structural response, the nonstructural component is treated as structural, and the relevant structural provisions apply. For example, a nonstructural masonry infill wall is regarded as structural and therefore within the scope of Chapter 7. The nonstructural component, such as cladding or heavy partitions, would typically affect the structure's response by means of its connections to it and the stiffening or damping effect it provides. The interaction may be beneficial or detrimental depending on location. Partial infill between columns with masonry walls may create a short column effect, i.e., reduce the effective length of the column, and seriously affect the structural response.

Nonstructural components are regarded as deformation-sensitive when they are affected by the structure's deformation, typically measured by inter-story drift. For example, a stud and plaster partition, connected from floor to floor or between structural walls or columns, can be damaged by racking caused by building drift.

A recurring problem in earthquakes has been the jamming of large overhead doors in fire stations, causing delay in dispatching fire apparatus. Excessive structural drift causes the support and guide rails to distort and the door to bind. Excessive drift has also caused doors opening onto exit corridors to jam, trapping the occupants. In both these instances, the remedy lies in controlling structural drift, rather than nonstructural design measures.

When there is no structural-nonstructural interaction because of the imposed deformation problem, the nonstructural component is regarded as acceleration-sensitive. An example is an item of mechanical equipment located on a building floor. Since an item on an upper floor might incur greater forces because of its location, the force equation accounts for this. Nonstructural components of large mass—for example, large water tanks—can also affect structural response, and must be considered in estimating loads.

C11.5.2 Base Isolation

Nonstructural components that cross the isolation interface of a base-isolated structure must be designed to accommodate the large potential relative displacements that may occur. These relative displacements may exceed one foot in length, and special detailing may be necessary. Swivel joints in piping and large flexible joints in ductwork may be necessary. Stairs must be attached to one side of the interface and allowed to move freely over the other. Elevator shafts may be attached to the superstructure and allowed to project down below the interface, with no attachment below the interface level. Special detailing is necessary for architectural components that cross the interface; in some instances, sacrificial components or materials may be used that are replaced after a seismic event of sufficient magnitude to damage them.

C11.6 Acceptance Criteria for Acceleration-Sensitive and Deformation-Sensitive Components

Acceptance criteria are provided for each nonstructural component or component group, to establish conformance with Performance Levels. The first level with well-defined meaning with reference to nonstructural components is the Life Safety Performance Level, because Collapse Prevention is defined only in structural terms and the Hazards Reduced Level has no specified nonstructural requirements. In the Immediate Occupancy Performance Level, which exceeds the Life Safety Performance Level, the requirements may be either the same as, or much stricter than, those for Life Safety, depending upon the component.

Where anchorage or another rehabilitation method for a component to achieve Life Safety prevents functional damage as well, higher criteria may also be met. (The level of motion, and thus forces resisted, is a function of the ground motion criteria chosen, which is only specified by the *Guidelines* for the BSO). In other cases, the criteria become much more demanding as the level increases. Thus, precast concrete exterior cladding panels might meet acceptance criteria for the Life Safety Performance Level, but fall short of the criteria for Immediate Occupancy, because possible distortion and loss of weather protection might render the building unusable, even if panels do not fall.

In some instances, because of the nature of some nonstructural components, quantitative acceptance criteria are not justified, and qualitative statements are used. The intent is to limit the need for engineering analysis and design where simpler methods are effective.

C11.6.1 Acceleration-Sensitive Components

For acceleration-sensitive components, the force provisions given in Sections 11.7.3 and 11.7.4 are expected to result in design force levels sufficiently high (realistic) to meet the effective needs of all Performance Levels. Providing lower design force levels for lower Performance Levels may be ineffective, since nonstructural elements tend to require rehabilitation techniques such as bolts and braces that can be economically designed to be adequate for a wide range of accelerations; that is, the type and layout of the bracing or anchorage scheme is more critical to the success of a rehabilitation strategy than the design force applied to it. Consequently, a conservative design force is recommended for all Performance Levels in acceleration-sensitive elements, and will have little cost penalty because of the simple techniques (bolting and bracing) that are involved.

For heavy equipment mounted on upper floors or roof, it is recommended that Equations 11-2 and 11-3 be used, because these equations introduce the effects of amplification caused by height. It is suggested that heavy equipment mounted on the third floor or above be analyzed in this way, if the structure is flexible. Experience has shown that rooftop mechanical equipment at the third floor or over is susceptible to accelerations and may shift, causing expensive damage and probable loss of function.

C11.6.2 Deformation-Sensitive Components

For deformation-sensitive components, the deformation limits of the *Guidelines* represent, in an average case, deformations associated with severe nonstructural damage for the Life Safety Performance Level and moderate nonstructural damage for the Immediate Occupancy Performance Level.

The values for limiting structural drift ratios have been derived primarily from the NIBS *Loss Estimation Methodology* (RMS, 1995), and refer to mean estimates of actual (unreduced) drift. The values in this study are derived from test results and experience, but a single median threshold value is provided for all drift-sensitive components (RMS, 1995, Table 5A-3). In addition, median drift values for damage states are provided for drift-sensitive nonstructural components located in each of 35 building types (RMS, 1995, Table 5A-4). In this table, the median drift values vary primarily because of assumed differences in floor-to-floor height for the different building types. These median drift values are, in turn, related to calculated drift values for corresponding structures to produce fragility curves.

While the NIBS *Loss Estimation Methodology* probably represents the best attempt yet to establish drift values related to damage states, the use of a single median drift ratio value—based on very limited laboratory testing—as an acceptance criterion is a wide stretch in usage. It is suggested that the limiting drift ratio values shown in the *Guidelines* in Chapter 2 be used as guides 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 design of structural rehabilitation. These criteria should be regarded as a flag for the careful evaluation of the structural-nonstructural interaction and assessment of damage states, rather than the required imposition of an absolute acceptance criterion that might suggest costly redesign of the structural rehabilitation.

C11.6.3 Acceleration- and Deformation-Sensitive Components

Some components are both acceleration- and deformation-sensitive, but generally one or the other of these characteristics is dominant, as is suggested in Table C11-1. The engineer must use judgment in evaluating the need for rehabilitation and the appropriate design solution.

C11.7 Analytical and Prescriptive Procedures

The *Guidelines* establish the minimum rehabilitation procedures that relate to desired Performance Levels. Thus, where Analytical Procedures are required, Prescriptive Procedures do not apply. Where Prescriptive Procedures are permitted, Analytical Procedures may be used at the discretion of the engineer.

C11.7.1 Application of Analytical and Prescriptive Procedures

For nonstructural components, the Analytical Procedure, which consists of the Default Equation and the General Equation approaches, is applicable to any case. The Prescriptive Procedure is limited by Table 11-1 to specified combinations of seismicity and component type for compliance with the Life Safety Performance Level.

C11.7.2 Prescriptive Procedure

These procedures apply where established rehabilitation methods are defined, and analysis is not required beyond establishing weights and/or dimensions. In general, the detailed requirements can be established by reference, such as the Ceilings and Interior Systems Construction Association (CISCA) standards for suspended ceilings, or the SMACNA standards (1980, 1982, 1991) for support of ductwork and piping. It may be necessary to specify different parts of these standards as applicable to different Rehabilitation Objectives, depending on the relevant Seismic Zone. Assessment of these components involves checking whether the component is braced or attached per prescriptive requirements.

Also found in the sections for individual components is guidance on the application of separately promulgated and published references so that they can be consistently and compatibly used with the requirements of this document. In most cases, these references do not specifically refer to seismic issues. Thus, translation of Seismic Zone definitions, consideration of Performance Levels as they relate to the objectives underlying a given standard or reference, and other conversions and adaptations are often necessary.

C11.7.3 Analytical Procedure: Default Equation

The Analytical Procedure includes two methods: one is defined by Equation 11-1, the other by Equations 11-2 and 11-3. These equations are derived from the proposed *1997 NEHRP Recommended Provisions* (BSSC, 1997). In these *Provisions*, the second two equations are shown as alternates, but for this document Equation 11-1 fills the role of a simple default equation that gives conservative results, and Equations 11-2 and 11-3 provide more detailed equations that will give a more precise and generally less conservative result. For nonstructural components, the use of the Default Equation that provides for conservative force levels is unlikely to carry a cost penalty; many acceleration-sensitive components can be easily rehabilitated by simple anchoring and bracing, and designing these for larger forces will generally be more cost-effective than using a more complex Analytical Procedure.

C11.7.4 Analytical Procedure: General Equation

The use of Equations 11-2 and 11-3 to determine the forces for acceleration-sensitive components will give a more precise and generally less conservative result. For components such as heavy cladding, where connections are critical, the more precise Analytical Procedure should always be used. The expanded equation also allows for derivation of force levels to meet lower as well as higher ground motion criteria, and might, in some circumstances, result in a more economical solution. All equations were adapted from similar equations in the *NEHRP Recommended Provisions* (BSSC, 1997).

C11.7.5 Drift Ratios and Relative Displacements

For some deformation-sensitive components, where drift limits are specified as part of the acceptance criteria, the building drifts that relate to the location of these nonstructural components must be estimated and compared to the acceptance levels. Equations 11-4 and 11-5 are used for this analysis. If the drift acceptance criteria are not met, engineering judgment must be used to determine the relative economies of reducing the building drift compared to changing the nonstructural component or detailing it to accept the level of drift.

C11.7.6 Other Procedures

Nonstructural components attached to the roof, floors, walls, or ceilings of a building (such as mechanical equipment, ornamentation, piping, and partitions) respond to the building motion in much the same manner that the building responds to the ground motion. However, the building motion may vary substantially from the ground motion. The most common method of representing nonstructural support excitation is by means of roof and floor response spectra at the nonstructural support locations derived from the dynamic analysis of the building.

The development of site-specific ground motions, expressed as site-specific response spectra or acceleration time-histories, is discussed in Section 2.6.2. The use of site-specific ground motions in alternative analytical procedures would require the conversions of these site-specific ground motion parameters to building floor and roof response spectra or acceleration time-histories at the support locations of the nonstructural components.

Floor and roof response spectra can be computed most directly from a dynamic analysis of the structure conducted on a time-step-by-time-step basis using site-specific acceleration time-histories. According to Section 2.6.2, Site-Specific Ground Shaking Hazard, at least three time-histories (for each component of motion) should be used.

Nonstructural components that are supported at multiple locations throughout the building could have different floor or roof spectra for each support location. The relative displacement between supports should be considered in the evaluation of the nonstructural component's performance. There are complex analytical techniques available to calculate these relative displacements, using different spectra at each support location or using different input time-histories at each different support. Careful consideration must be given to the fact that the maximum response at various support locations might not occur at the same time.

For determining Life Safety and Immediate Occupancy Performance Levels for nonstructural components, the time-consuming and costly analytical procedures outlined above are not as cost-effective as the Prescriptive and Analytical Procedures presented in Section 11.7. Recent research by Drake and Bachman (1995), using a sample of 405 buildings and events, indicates that Sections 11.7.3 and 11.7.4, which are based on the Analytical Procedures in 1997 *NEHRP Provisions* (BSSC, 1997), will provide a reasonable upper bound for the seismic forces on the nonstructural components wherever they are located in the building. Therefore, complex analysis methods used for the structural and nonstructural components are not necessary for the evaluation and rehabilitation of typical building nonstructural components covered in Chapter 11.

C11.8 Rehabilitation Concepts

A general set of alternative methods is available for the rehabilitation of nonstructural components. These are briefly outlined in this section, in approximate order of their cost and effectiveness, together with examples of each to clarify the intent of this classification. However, the choice of rehabilitation technique and its design is the province of the design professional, and the use of alternative methods to those noted below or otherwise customarily in use is acceptable, provided it can be shown to the satisfaction of the building official that the acceptance criteria can be met.

C11.8.1 Replacement

Replacement involves the complete removal of the component and its connections, and its replacement by new components; for example, the removal of exterior cladding panels, the installation of new connections, and installation of new panels. As with structural components, the installation of new nonstructural components as part of a seismic rehabilitation project should be the same as for new construction.

C11.8.2 Strengthening

Strengthening involves additions to the component to improve its strength to meet the required force levels; for example, additional members might be welded to a support to prevent buckling.

C11.8.3 Repair

Repair involves the repair of any damaged parts or members of the component, to enable the component to meet its acceptance criteria; for example, some corroded attachments for a precast concrete cladding system might be repaired and replaced without removing or replacing the entire panel system.

C11.8.4 Bracing

Bracing involves the addition of members and attachments that brace the component internally and/or to the building structure. A suspended ceiling system might be rehabilitated by the addition of diagonal wire bracing and vertical compression struts.

C11.8.5 Attachment

Attachment refers to methods that are primarily mechanical, such as bolting, by which nonstructural components are attached to the structure or other supporting components. Typical attachments are the bolting of items of mechanical equipment to a reinforced concrete floor or base.

Supports and attachments for mechanical and electrical equipment should be designed according to good engineering principles. The following guidelines are recommended.

1. Attachments and supports transferring seismic loads should be constructed of materials suitable for the application, and designed and constructed in accordance with a nationally recognized structural code.

2. Attachments embedded in concrete should be suitable for cyclic loads.

3. Rod hangers may be considered seismic supports if the length of the hanger from the supporting structure is 12 inches or less. Rod hangers should not be constructed in a manner that would subject the rod to bending moments.

4. Seismic supports should be constructed so that support engagement is maintained.

5. Friction clips should not be used for anchorage attachment.

6. Expansion anchors should not be used for mechanical equipment rated over 10 hp, unless undercut expansion anchors are used.

7. Drilled and grouted-in-place anchors for tensile load applications should use either expansive cement or expansive epoxy grout.

8. Supports should be specifically evaluated if weak-axis bending of cold-formed support steel is relied on for the seismic load path.

9. Components mounted on vibration isolation systems should have a bumper restraint or snubber in each horizontal direction. The design force should be taken as $2F_p$.

10. Oversized washers should be used at bolted connections through the base sheet metal if the base is not reinforced with stiffeners.

Lighting fixtures resting in a suspended ceiling grid may be rehabilitated by adding wires that directly attach the fixtures to the floor above, or to the roof structure to prevent their falling.

C11.9 Architectural Components: Definition, Behavior, and Acceptance Criteria

C11.9.1 Exterior Wall Elements

C11.9.1.1 Adhered Veneer

A. Definition and Scope

This section refers to veneer that relies for its support on adhesive attachment to a backing or substrate rather than mechanical attachments. The section covers both thin units that provide a weather-resistant exterior surface, and exterior plaster (stucco) that is applied in one or more coats to the supporting substrate. Four categories of veneer are identified in the *Guidelines*.

B. Component Behavior and Rehabilitation Concepts

The typical failure mode is cracking of the adhered veneer and/or separation and falling from the backing. Any separation of the surface veneer from its substrate is critical, because it concentrates loads on areas surrounding the separation and provides a place for the weathering elements to penetrate and cause progressive failure.

The adherence of the veneer to its support substrate is generally covered by prescriptive requirements that are not specifically seismically related. For walls that do not fall within the limitations of conventional light frame construction, the supporting elements must be reviewed analytically to determine the likelihood of producing deformations that might detach the veneer materials.

The possibility of a threat to life safety depends on the height of the veneer, the level of use of adjoining areas by personnel, and the size or weight of fragments that could possibly fall from the wall. There is a distinction between the displacement (falling) of areas of veneer and the falling of individual units such as tiles. All of these factors must be evaluated in order to make a determination.

The replacement of adhered veneer that is cracked or partially separated from its substrate may be very costly. For architectural reasons it may be necessary to replace much larger areas than those that are actually vulnerable, because of the difficulty in matching new and old surfaces.

In some cases, substantial damage to the adhered veneer may be temporarily allowed while declaring the building to be ready for immediate occupancy. This will be the case when possible progressive separation of the veneer will not pose a threat to personnel using the building, and when the damage does not allow penetration of weather elements in a way that would prevent, or limit, the use of the building. The full range of options available to the client must be spelled out so that an economic determination may be made.

Critical locations for evaluation of the veneer are those where substantial deformation is possible and where discontinuity in the surface exists, such as around openings, and especially at corners. The evaluation of possible potential damage will include the existence, or lack, of reinforcing around these discontinuities and the amount of deformation that will be allowed based on analysis of the structure's deformation characteristics.

A description of Adhered Veneer Categories 1, 2, and 3 and typical structural backing may be found in MIA (1994). Information regarding nonstructural exterior plaster may be found in a reference of the Portland Cement Association (PCA, 1995).

C. Acceptance Criteria

The limiting structural drift ratio of 0.030 for the Life Safety Performance Level represents extensive damage for drift-sensitive components in the NIBS Loss Estimation Methodology (RMS, 1995) The limiting drift ratio of 0.010 for the Immediate Occupancy Performance Level represents moderate to extensive damage in the drift-sensitive components. These limits must be carefully evaluated by the engineer with respect to the estimated structural drifts and the detail of the veneer substrate and its relation to the structure.

Compliance with acceptance criteria is intended to achieve the following performance:

Life Safety Performance Level. Cracking of any extent and some detachment in noncritical areas may occur.

Immediate Occupancy Performance Level. Some cracking and detachment of a few individual pieces in noncritical areas may occur.

C11.9.1.2 Anchored Veneer

A. Definition and Scope

This section identifies the distinguishing feature of this veneer to be the mechanical attachments and defines three categories of veneer that are included. In addition, the critical function of the mechanical fasteners is described. Prescriptive values for mechanical connectors are available from the manufacturers.

Proper identification of anchored veneer is important. It is often difficult to establish if the anchors are present. In many older buildings with multiwythe walls, a single wythe of facing brick is placed on the exterior without physical connections such as headers or anchors. While this is more likely to be considered a multiwythe, unreinforced masonry wall, the possibility exists for the separation of the entire wythe of brick from the structural wall. Where this occurs, the exterior wythe must be anchored or removed.

B. Component Behavior and Rehabilitation Concepts

Failure occurs by separation or distortion of the unit in relation to its supporting structure, brought about by pulling out, distortion, or buckling of the mechanical fasteners.

The possibility of a threat to life safety depends on the height of the veneer, the possibility of the use of adjoining areas by personnel, and the size or weight of fragments that could possibly fall from the wall. All of these factors must be evaluated in order to make a determination.

Cracking of units, in a way that does not adversely affect the attachment of the units to the structural backing, is considered to be a Damage Control Performance Range problem. As soon as the damage becomes a factor in the mechanical attachment and the veneer in question is over four feet above the adjacent floor or ground, and is in an area that is likely to be occupied, it becomes a Life Safety question.

Distinction must be made between damage that occurs to the units only, and that which affects or may affect the mechanical fasteners that support the units.

As with adhered veneer, critical locations for evaluation of the veneer are those where substantial deformation is possible and where discontinuity in the surface exists, such as around openings, and especially at corners. The evaluation will include the existence or lack thereof of reinforcing around these discontinuities, and the amount of deformation that will be allowed based on analysis of the structure's deformation characteristics.

A description of the three types of anchored veneer and their typical structural backing may be found in MIA (1994) and ASTM (1995).

C. Acceptance Criteria

The limiting structural drift ratio of 0.020 for the Life Safety Performance Level represents extensive damage for drift-sensitive components in the NIBS *Loss Estimation Methodology* (RMS, 1995). A more restrictive drift criterion is selected for this component compared to adhered veneer because of the generally larger, and potentially more life-threatening, components and materials that are used in these exterior systems. The limiting drift ratio of 0.010 for the Immediate Occupancy Performance Level represents moderate to extensive damage in the drift-sensitive components. These limits must be carefully evaluated by the design professional with respect to the estimated structural drifts, the detail of the veneer substrate, and its connection to the structure.

Compliance with acceptance criteria is intended to achieve the following performance:

Life Safety Performance Level. Cracking of the masonry units may occur as long as it does not significantly affect the load distribution on the anchors. Failure of anchor elements that result in falling of units that are more than four feet above the ground or adjacent exterior area may not occur unless adjacent areas are inaccessible to pedestrians and all vehicles.

Immediate Occupancy Performance Level. Some cracking of masonry units is acceptable, but substantial weather protection must be maintained.

D. Evaluation Requirements

No commentary is provided for this section.

C11.9.1.3 Glass Block Units and Other Nonstructural Masonry

A. Definition and Scope

No commentary is provided for this section.

B. Component Behavior and Rehabilitation Concepts

This section refers to the generally single-wythe glass block and other masonry units that are self-supporting from a vertical load standpoint and, to a limited extent, for lateral forces—as long as very conservative height-to-thickness ratios are maintained—but which cannot resist forces imposed from other elements of the building nor significant differential deformations.

Failure occurs by cracking of the mortar joints or units and lateral displacement along those cracks. Hairline cracks due to shrinkage or small movements of the supporting structure are generally not critical. However, cracks over three to five mils (three to five thousandths of an inch, 0.007-0.012 mm.), or any cracks showing lateral displacement, signify a loss of shear capacity along that line and therefore indicate failure.

Prescriptive requirements for glass block units should be used as the criteria for rehabilitating these walls. These prescriptive requirements include type, strength of mortar, reinforcing of the joints with galvanized steel wires, limitations on the size of the panels, and the need for properly filled expansion joints around properly sized areas of panel. Refer to the 1994 *Uniform Building Code* (*UBC*), Section 2110 (ICBO, 1994) for specific prescriptive requirements that may be used. This document does not link itself to any of the model codes in use in the United States, and is, in general, coordinated with the *NEHRP Provisions for New Buildings* (BSSC, 1995), but in this case only the *UBC* reference is applicable. For walls larger than 144 square feet, analysis of drift and forces is necessary, and careful engineering design of the wall is required.

For Life Safety, the same general criteria exist for these as for other masonry units: consideration of the height of the wall, the weight of material that could fall, and the possibility of people being in the adjacent areas.

These walls should be replaced if their installation and condition significantly differ from the prescriptive requirements of current building codes, and their location is critical with respect to Life Safety.

C. Acceptance Criteria

The limiting structural drift ratio of 0.020 for the Life Safety Performance Level represents extensive damage for drift-sensitive components in the NIBS *Loss Estimation Methodology* (RMS, 1995). The limiting drift ratio of 0.010 for the Immediate Occupancy Performance Level represents moderate to extensive damage in the drift-sensitive components. These limits must be carefully evaluated by the engineer with respect to the estimated structural drifts, the detail of the glass block, and its connection to the structure.

Compliance with acceptance criteria is intended to achieve the following performance:

Life Safety Performance Level. Hairline cracking may occur so long as the shear strength and out-of-plane bending strength of the wall are not significantly impaired. Displacement of some units may occur in noncritical areas.

Immediate Occupancy Performance Level. Hairline cracking may occur so long as the shear strength and out-of-plane bending strength of the wall are not significantly impaired. No displacement of units may occur.

D. Evaluation Requirements

No commentary is provided for this section.

C11.9.1.4 Prefabricated Panels

A. Definition and Scope

This section encompasses types of exterior panels that generally span from floor to floor or column to column and are manufactured to quality control standards, ensuring the unit has a minimum defined strength. Type 1 (precast concrete panels) and Type 2 (metal faced insulated panels) are, in effect, large building blocks that are capable of structurally withstanding the forces applied within the perimeter connections. Type 3 (steel strong-back panels with mechanically attached facings) is made up of structural elements that can be designed using typical structural analysis procedures for the materials and concept involved.

B. Component Behavior and Rehabilitation Concepts

This section defines the two different categories of failure that might occur. One is the failure of the unit itself, due to improper design or defective manufacture for the loads (primarily racking) resisted within the panel. The second mode pertains to the connecting elements that attach the panels to the building's structural system, which may fail due to either acceleration-based forces, or their inability to withstand the deformation of the structure. Criteria for new prefabricated panels require a considerable multiplier on design loads for connections, to limit the possibility of connection failure.

Often these panels must be replaced for nonseismic reasons, if their condition is such as to make them non-weather-resistant or unsightly. The possibility of some damage or cracking, if minor, under design seismic forces and deformations may, under some circumstances, limit the life of the panel but not create an immediate need for replacement for Life Safety reasons.

On upper floors of buildings, the loss of strength in the connections of these panels will create a continuing life safety problem for anyone adjacent to the building. Panels that are not hazardous may nevertheless suffer seismic damage, such as cracking or displacement, that will diminish weather and thermal resistance and limit the use of adjacent space, unless temporary covering is acceptable to meet Immediate Occupancy requirements.

The panels must be evaluated for their ability to act as the building envelope in the case of damage such as cited above. Consideration must be given to the ductility of concrete anchors where they might be used to provide attachment of the panels to the main structure. If the anchor does not exhibit adequate ductility to preclude a brittle failure, then greater care must be take in evaluating the strength of the attachments under assumed forces.

C. Acceptance Criteria

The limiting structural drift ratio of 0.020 for the Life Safety Performance Level represents extensive damage for drift-sensitive components in the NIBS *Loss Estimation Methodology* (RMS, 1995). The limiting drift ratio of 0.010 for the Immediate Occupancy Performance Level represents moderate to extensive damage in the drift-sensitive components. These limits must be carefully evaluated by the engineer with respect to the estimated structural drifts, the detail of the panels, and their connection to the structure.

Compliance with acceptance criteria is intended to achieve the following performance:

Life Safety Performance Level. Considerable cracking and detachment of the units may occur, as long as the panels remain in place. Detachment of weather stripping may occur.

Immediate Occupancy Performance Level. Some cracking and detachment of the units may occur, as long the panels remain in place. Minimal detachment of weather stripping may occur.

D. Evaluation Requirements

No commentary is provided for this section.

C11.9.1.5 Glazing Systems

A. Definition and Scope

No commentary is provided for this section.

B. Component Behavior and Rehabilitation Concepts

Metal frames and mullions that are attached to a structure subject to large deformations will flex and twist, pulling the frame from the glass in one direction. In the return motion, the glass may be out of the frame—the result is instantaneous damage. Division bars in any system are suspect; they are seldom anchored, and even when they are, the metal often does not have enough strength to resist the twisting effect. In glazing systems where the supporting frames remain undamaged, yet the glass is damaged or has fallen out, there are four conditions that may prevail.

1. The glass is cut too small for the opening: not enough edge “bite.”

2. There is no edge blocking, causing the glass to shift too far to one side.

3. The glass is cut too large for the opening, leaving no room for expansion (inadequate edge clearance).

4. Roll-in vinyl gaskets that fall from the opening allow the glass to slide back and forth in the opening, causing shattering or falling. These gaskets create the pressure that holds snap-on stops in the opening.

Safety is also affected by the type of glass. When broken, ordinary annealed glass produces sharp-edged shards that can cause serious injury. Code provisions implemented in the 1970s now require safety glass (such as tempered, wired, or laminated glass) when the glass extends to within 18 inches of the ground or floor. Tempered glass fractures into small round-edged pieces that are significantly less dangerous than shards, and this type of glass up to ten feet in height does not represent a significant life safety threat. Laminated glass generally remains intact even if it cracks.

Guidelines on the general analysis and design of glazed walls can be found in the *Aluminum Design Guide Curtain Wall Manual* (AAMA, 1996a) and *Rain Screen Principle and Pressure Equalized Wall Design* (AAMA, 1996b).

As indicated in the definition and scope section, the evaluation of these panels must consider both the structural support provided by the mullions and the other supporting members, as well as the containment of the glazing and the method of doing that within the supports. Wherever possible, consideration should be given to converting the method of enclosure to wet glazing, which has proven to be durable, and much tougher in resisting dynamic loads than other methods.

C. Acceptance Criteria

The limiting structural drift ratio of 0.020 for the Life Safety Performance Level represents extensive damage for drift-sensitive components in the NIBS *Loss Estimation Methodology* (RMS, 1995). The limiting drift ratio of 0.010 for the Immediate Occupancy Performance Level represents moderate to extensive damage in the drift-sensitive components. These limits must be carefully evaluated by the engineer with respect to the estimated structural drifts, the detail of the glazing, and its connection to the structure.

Compliance with acceptance criteria is intended to achieve the following performance:

Life Safety Performance Level. Considerable loss of weather stripping may occur. Shattering of glass or material falling out from more than four feet above interior floors or adjacent exterior area may not occur.

Immediate Occupancy Performance Level. Some limited loss of weather stripping may occur.

D. Evaluation Requirements

No commentary is provided for this section.

C11.9.2 Partitions

C11.9.2.1 Definition and Scope

Partitions are categorized as "heavy" or "light"; the intent is to distinguish between masonry or other heavy assemblies, and typical replaceable partitions consisting of metal or wood studs with a layer of gypsum board on each side. These lightweight assemblies weigh approximately five pounds per square foot, which establishes the category definition of "heavy" or "light".

Full-height glazed walls are similar to exterior glazing in assembly and so are required to meet the requirements of these systems.

C11.9.2.2 Component Behavior and Rehabilitation Concepts

If heavy partitions are isolated from the structure by providing a continuous gap between partition and surrounding structure, or are freestanding, the partitions will be acceleration-sensitive and should be analyzed independently to meet the material acceptance requirements (e.g. Section 7.4 for masonry). In some instances, wood stud partitions and facings may be of sufficient mass to interact with the structure and should be analyzed as structural.

In some structural types, wood frame partitions may be enhanced to become shear panels, and must be analyzed as structural elements. Partitions that span from floor to floor (roof) or floor to ceiling are deformation-sensitive. Partitions that are freestanding or span to a light metal grid hung ceiling are acceleration-sensitive in all directions. Deformation-sensitive lightweight partitions loaded in-plane can be subjected to:

1. Minor shear cracking

2. Major shear cracking and deformation at attachments to structure, with dislodgment of some applied finish materials

3. Distortion and fracturing of partition framing, and detachment and fracturing of the surface materials

Since partitions are both acceleration- and deformation-sensitive, drift analysis is required for rehabilitating partitions to meet or exceed the Life Safety Performance Level, because partition in-plane deformations must be known. For Immediate Occupancy and Operational Performance Levels, all partitions are candidates for rehabilitation. Noncritical lightweight partitions may be treated as replaceable in many instances where life safety is not an issue and special detailing is not cost-beneficial.

Heavy infill partitions should be rehabilitated according to the provisions of Chapter 7. Heavy free-standing partitions that cannot meet the force and/or displacement requirements of Section 11.7.4 will probably need to be replaced with lightweight partition materials.

Heavy partitions that can meet out-of-plane but not in-plane requirements, because they act as infill, may be rehabilitated by detailing that detaches the partitions from the surrounding structure (provided the building structure is not adversely affected by this measure). For this method of rehabilitation, a detail must be introduced that retains restraint against out-of-plane movement of the partition, since it is no longer assisted by support from the surrounding structure. Judgment must be used to determine where lightweight partitions should be detached from surrounding structure to permit differential movement. Fire wall partitions forming part of the building fire safety system that are detached from the structure must be detailed substantially to retain their fire separation capability.

C11.9.2.3 Acceptance Criteria

The limiting structural drift ratio of 0.010 for the Life Safety Performance Level represents moderate to extensive damage for drift-sensitive components in the NIBS *Loss Estimation Methodology* (RMS, 1995). This represents a more restrictive criterion than for other drift-sensitive components, because falling of heavy partition components in interiors represents a considerable life safety threat. In practice, it may be more cost-effective to replace heavy partitions with steel stud and gypsum board walls, but if the building interior is of historical significance this solution may not be acceptable. The limiting drift ratio of 0.005 for the Immediate Occupancy Performance Level represents negligible to moderate damage in the drift-sensitive components. These limits must be carefully evaluated by the engineer with respect to the estimated structural drifts, the details of the walls, and their relationship to the structure.

To confirm that the acceptance criteria are met, in addition to the required Analysis Procedures, the adequacy of the following applicable partition components must be inspected and assessed:

1. The attachment of the finish materials to the partition
2. The condition at the top of the partition, particularly as to whether or not there is a connection to the building floor or roof structure, ceiling system, and the like
3. The connection at the top of the partition (if any) to allow for the vertical deflection of the structure above, to resist out-of-plane seismic forces, and to accommodate in-plane inter-story drift displacements
4. The connection at the bottom of the partition to the building floor to resist the in-plane and out-of-plane seismic forces on the partitions
5. The partition support elements (such as wood or metal studs and solid or hollow unit masonry) to resist the in-plane and out-of-plane seismic forces and inter-story displacements

Compliance with acceptance criteria is intended to achieve the following performance:

Life Safety Performance Level. Typical damage to light partitions is that of cracking and distortion; this is not categorized as a life safety issue, based on experience in earthquakes. In URM buildings, light frame partitions, although poorly constructed and damaged, have often succeeded in supporting damaged floors and roofs despite being theoretically quite inadequate for such a purpose. This particularly applies to residential buildings, which have a high intensity of partitions in relation to floor area. The Coalinga, California earthquake of 1983 showed many instances of this phenomenon.

For heavy masonry or hollow tile partitions, some cracking and some displacement in noncritical locations may occur. Heavy partition assemblies, particularly if used as backing for ceramic or natural stone facing, may suffer some cracking, but must be carefully evaluated against the possibility of complete collapse or shedding large fragments.

Immediate Occupancy Performance Level. Minor cracking may occur in both light and heavy partitions; no heavy partitions may be displaced.

C11.9.2.4 Evaluation Requirements

No commentary is provided for this section.

C11.9.3 Interior Veneers

C11.9.3.1 Definition and Scope

Interior veneers are decorative finishes applied primarily to interior walls, both structural and nonstructural. Heavy veneers of natural stone or marble are common in entrances, elevator lobbies, and monumental staircases of major public buildings. Veneers, such as ceramic tile, are sometimes attached to ceilings. Wood veneers are sometimes used as wall (and occasionally ceiling) paneling, or as decorative cover to columns, but their light weight results in little inherent hazard, so they are not specifically identified in the *Guidelines*.

C11.9.3.2 Component Behavior and Rehabilitation Concepts

The particular concern with interior veneers relates to the possible falling hazard of heavy veneers in heavily occupied locations. Veneers are predominately deformation-sensitive, and if their backing becomes deformed their attachment may fail, particularly if the attachment is direct. Interior veneer seismic behavior depends on:

1. Its weight and height
2. The adequacy of the connection of the interior veneer to the backup support system

3. The adequacy of the backup support system and its connection to the structure to resist the out-of-plane and in-plane seismic forces, and in-plane inter-story drift of the structure

Adhered interior veneer reflects the seismic performance of the backup system. If the rigid backup masonry or concrete walls crack, these cracks will be reflected in the interior veneer. The strength and stiffness of the structure, as well as the backup system, and their compatibility with the inherent strength of the veneer must be considered.

Drift analysis is required for rehabilitating interior veneer to meet or exceed the Life Safety Performance Level, because in-plane deformations of the backup support system must be determined. Only heavy veneer located higher than four feet from the floor need be considered for the BSO.

To confirm that the acceptance criteria are met, in addition to the required Analysis Procedures, the engineer shall inspect, and assess the adequacy of, the following applicable components of the interior veneer:

1. The attachments and connections (e.g., mortar, adhesive, wires) of the interior veneer to the backup system (e.g., metal or wood studs, solid or hollow unit masonry, or reinforced concrete)

2. The adequacy of the backup support system and its connection to the building structure to resist the out-of-plane and in-plane seismic forces, and in-plane inter-story drift

Because interior veneers are, by nature, a visually important and decorative element, the rehabilitation methods must take into account the resulting rehabilitated appearance of the veneer.

Before replacement/resetting of the interior veneer, the backup support system and building structural system shall be examined and analyzed for their ability to withstand the design seismic forces from, and displacements with, the allowable drift limitation of the interior veneer. Unlike exterior veneers, corrosion is not likely to affect mechanical fastening systems, unless water leakage has been present. After this analysis, consideration should be given to the possibility that the particular interior veneer is not compatible with the stiffness of the rehabilitated building structural system. If this is so, a determination must be made whether to replace the interior veneer and/or backup system with different materials and/or systems, or rehabilitate the main structural system to meet the acceptable drift limitations of the replaced/reset or new veneer material system. The latter action may be unrealistic in structural or economic terms unless there are additional reasons for meeting specific drift criteria.

C11.9.3.3 Acceptance Criteria

The limiting structural drift ratio of 0.020 for the Life Safety Performance Level represents extensive damage for drift-sensitive components in the NIBS *Loss Estimation Methodology* (RMS,1995). The limiting drift ratio of 0.010 for the Immediate Occupancy Performance Level represents moderate to extensive damage in the drift-sensitive components. These limits must be carefully evaluated by the engineer with respect to the estimated structural drifts and the detail of the veneer substrate and its relation to the structure.

Compliance with acceptance criteria is intended to achieve the following performance:

Life Safety Performance Level. Some cracking and displacement of a few units may occur.

Immediate Occupancy Performance Level. Minor cracking, but no displacements, may occur.

C11.9.3.4 Evaluation Requirements

No commentary is provided for this section.

C11.9.4 Ceilings

C11.9.4.1 Definition and Scope

Section 11.9.4.1 defines the main types of ceilings typically found in existing buildings. The chief distinction is between those that are attached directly to the building structure, and those that are suspended below the structure by wires or other attachment systems.

C11.9.4.2 Component Behavior and Rehabilitation Concepts

The seismic behavior of ceilings is primarily influenced by the seismic performance of their support systems. Surface-applied ceiling finishes usually perform well. Suspended metal lath and plaster ceilings perform well if properly braced, and if the adhesion of the plaster to the lath, which deteriorates with age, is still effective. Suspended integrated ceiling systems are highly susceptible to damage unless properly braced and detailed.

This section describes the typical behavior of the variety of ceiling types, with emphasis on the high susceptibility of modern suspended integrated ceilings that have not been braced with splay wires and vertical compression struts.

Surface-applied acoustical tile, plaster, or gypsum board perform well, provided the surface to which these materials are attached does not crack or spall. Ceiling tile can fall due to adhesive failure. Plaster on wood or metal lath attached to wood framing may not perform well. Plaster may have fine cracks that could lead to spalling, particularly along the wood lath. Large areas of fallen plaster in stairways and corridors could impair routes of ingress and egress.

Gypsum board ceilings properly applied—directly to the bottom of wood joists or suspended from wood joints with short wood hangers—will perform well, because the gypsum board is inherently rigid in the plane of the ceiling. Metal lath and plaster ceilings perform well, provided they are laterally braced, the hanger wires are properly connected to the structure above, and metal lath is properly wired to the furring channels. Hanger wires may unwind and pull through their connections or break, or their connections to the structure may fail.

Suspended integrated ceiling systems are highly susceptible to damage, unless they are braced with splay wires and vertical compression struts. Earthquakes cause unbraced ceilings to swing on their hanger wires and pound against, or come off, their supports on adjacent partitions and walls. These suspended ceilings are also subjected to pounding forces from light fixtures, ceiling ventilation diffusers, sprinkler heads, and partitions, which damage the ceiling support members and panels. Ceiling systems that are flexible in the plane of the ceiling (lay-in and concealed spine) may sustain greater damage than systems with greater in-plane rigidity (metal lath and plaster, and gypsum board).

Lightweight grid/panel systems in commercial buildings such as stores and supermarkets are very susceptible to damage because these structures often suffer major deformations. Displacement and falling of lightweight ceiling tiles and the grid, although it causes much disruption and is costly to replace, is not in itself a life safety threat, and a good educational program of self-protection is likely to be a much more effective—and cost-effective—way of preventing injury than bracing an existing ceiling of this type. However, heavy items supported by the ceiling, such as lighting fixtures and air diffusers, must have an independent support that prevents their falling if the supporting grid falls or is badly distorted. Suspended ceilings in certain occupancies—such as in hospital rooms or at exit doors and lobbies—may, however, require special attention with respect to maintaining life safety.

Ceiling systems are both acceleration- and deformation-sensitive. Deformation of the diaphragm may cause horizontal distortion of a ceiling, and deformation of a vertical structure may cause the ceiling to lose its perimeter support and drop. Category a and b ceiling rehabilitation assumes that the structural backing to which the ceiling is applied has been accepted or rehabilitated as part of the structural evaluation. Inspection of the ceiling materials and attachment will determine whether they should be repaired or replaced.

Commonly used industry installation details and procedures are available for the various materials and methods involved; these will not vary with the Performance Levels desired. Category c ceilings may include large ceilings of considerable weight, e.g., in auditoria and theaters, and so a careful force and displacement analysis is necessary. Heavy ceilings of this type can be a major threat to life safety. Category d ceilings, of simple configurations, are normally installed to code and industry standards based on prescriptive details and procedures, and no analysis is required. Special ceilings—of large area, unusual configuration, or with a large space between ceiling and floor or roof above—may require special engineering and analysis.

Ceilings (Categories a and b) that are directly or closely attached to the structure depend on their attachment for seismic integrity and, if properly installed and well maintained, generally meet acceptance criteria for all performance levels without difficulty. If the supporting structure fails, the ceiling materials will also fail. Suspended ceilings (Categories c and d) also interact closely with the structure and if the structure deforms severely, ceiling elements are almost certain to fall. Rehabilitation methods are aimed at ensuring acceptable performance under the structure's forces and drifts, within the structure's acceptance range. For tall long span structures, particularly steel moment frames, the amount of drift acceptable under structural Performance Level criteria may be difficult for the ceiling system to accommodate; in such structures, special design attention should be paid to ceiling rehabilitation.

For detailed evaluation, the ceiling category—a, b, c, or d—must be determined. The condition of the ceiling finish material and its attachment to the ceiling support system, the attachment and bracing of the ceiling support system, and the potential seismic impacts of other nonstructural systems on the ceiling system must be evaluated.

Although ceilings are drift-sensitive, no structural drift limits are stated in the *Guidelines*, because the complexities of structure/ceiling interaction make the identification of numerical values unrealistic. In general, lightweight integrated ceilings appear to experience the most damage in building types with long spans and flexible structural systems, such as commercial buildings—particularly retail stores. The limited testing of integrated ceiling installations that has been conducted has been inconclusive as to the value of compression struts, but tests have been conducted on small-scale ceiling systems. Such tests have indicated that these ceilings only failed at very high accelerations (e.g., 3.57g for a ceiling with no seismic restraints but perimeter attachment) but easily achieved drift ratios for the type of buildings noted above (0.625 inches—a drift ratio of 0.0035 for a 15-foot floor-to-floor height) (Anco, 1983). Much more testing of a variety of ceiling installations is necessary before definitive numerical values can be established, and it is also questionable whether the use of one or two variables such as drift or acceleration can determine ceiling performance.

Ceiling rehabilitation generally involves replacement, with either similar materials or more up-to-date alternatives. Ceilings, particularly modern integrated ceilings, generally have a relatively short life before they become aesthetically outdated. Thus, it is usually much more economical to replace the ceiling and at the same time update its appearance. Ceilings that brace lightweight partitions and mechanical and electrical components require special analysis and rehabilitation: it is generally preferable for seismic rehabilitation not to rely on the ceiling for bracing but to brace the partitions directly to the building structure. Heavy mechanical and electrical components should similarly be braced directly to the building structure.

Ceilings that brace partitions and/or mechanical and electrical components require special analysis and rehabilitation. Ceilings that cross building seismic and expansion joints require special attention. The rehabilitation procedure is to discontinue the ceiling system on each side of the joints. If the ceiling system must continue across the joint to satisfy HVAC, fire safety, or appearance requirements, the ceiling system must be modified to accommodate the relative structural movement allowed by the joint.

C11.9.4.3 Acceptance Criteria

Compliance with acceptance criteria is intended to achieve the following performance:

Life Safety Performance Level. For plaster ceilings, some cracking and displacement in noncritical locations may occur, but no falling of large ceiling areas (ten square feet or larger) weighing more than two pounds per square foot. For suspended ceilings, some loss of panels and distortion of grid may occur.

Immediate Occupancy Performance Level. For plaster ceilings, minor cracking and minor displacement in noncritical locations are permissible. Minor loss of panels and distortion of grid are allowed in suspended ceilings.

C11.9.4.4 Evaluation Requirements

No commentary is provided for this section.

C11.9.5 Parapets and Appendages

C11.9.5.1 Definition and Scope

Provisions for parapets are intended to apply primarily to unreinforced masonry parapets. Procedures for the design of unreinforced masonry (URM) walls are found in Chapter 7. Instances may occur where other types of parapet are not integral with, or properly attached to, the vertical building structure, and cantilever vertically above the roof structure.

C11.9.5.2 Component Behavior and Rehabilitation Concepts

Appendages are elements that are not integral with the building structure and cantilever vertically or horizontally from the structure. Critical issues for appendages are their weight, their attachment, their location—if over an entry or exit, public walkway, or lower adjacent buildings—and their surface area as a possible wind-sensitive item. These components may easily disengage and topple, and are among the most hazardous of nonstructural building elements in an earthquake. Because of the high possibility of casualties to people adjacent to buildings, rehabilitation of parapets and heavy appendages is required to meet the Collapse Prevention Performance Level.

Balconies generally involve an extension of the building floor structure, and should be evaluated as part of the structure. Eyebrows are cantilevered—or sometimes suspended—canopies over window openings, which may be continuous, or be separate elements over each window. Cornices are decorative elements at the top of a building that may sometimes be constructed of heavy masonry and cantilever a considerable distance, representing an obvious hazard to the public if inadequately designed and constructed.

In theory, falling of appendages might be permitted in inaccessible locations such as light courts, but in practice, all of these components should be rehabilitated; rehabilitation methods are relatively inexpensive and over the life of a building previously inaccessible locations might become accessible.

Appendages take a variety of forms, and their rehabilitation will depend on their characteristics and the nature of the structure to which they are attached. Because appendages are by nature exposed to the weather, they are very prone to corrosion and other material deterioration. Cornices may be the termination of a parapet and because of their location may present a particularly high risk. They may also be of great architectural significance, so the obvious rehabilitation measure of removing them may be unacceptable. Replacement by sheet metal or glass-reinforced plastic reproductions may be an appropriate seismic and economic solution, if the historic authenticity of the facade is permitted to be compromised.

C11.9.5.3 Acceptance Criteria

Compliance with acceptance criteria is intended to achieve the following performance:

Life Safety Performance Level. Components and elements may experience only minor displacement, except that they may fall into unoccupied areas.

Immediate Occupancy Performance Level. Components and elements may experience minor damage but no displacement of components or elements will occur.

C11.9.5.4 Evaluation Requirements

No commentary is provided for this section.

C11.9.6 Canopies and Marquees

C11.9.6.1 Definition and Scope

Canopies are horizontal, or near-horizontal, projections from an exterior wall, generally at a building entrance, to provide weather protection.

C11.9.6.2 Component Behavior and Rehabilitation Concepts

Canopies and marquees may become dislodged from their supports and collapse. On some occasions the failure of other appendages or exterior cladding may cause them to collapse.

These components may take the form of a horizontal extension of the structure (an overhang), in which case they should be analyzed as part of the building structure. Safety concerns for canopies apply to those that are attached to the building structure, and that sometimes are not part of the original structural design. Although defined as nonstructural, in the sense that they are not an integral part of the building structure, their evaluation and rehabilitation, if necessary, are a structural problem. Of particular concern are heavy canopies of reinforced concrete, with long cantilever spans designed to early seismic codes.

Canopies are sometimes designed as free-standing structures, associated with a building entrance, often with a distinctive architectural form with dramatic cantilevers; these also require a structural evaluation. Other canopies may be designed as propped cantilevers, suspended, or fully self-supporting, in which case they are defined as marquees. Marquees are typically temporary structures, such as tents, erected for special events, but the term is also used for freestanding structures covering a building entryway, which may be constructed of metal or glass or, for more formal buildings, reflect the construction and appearance of the building. Freestanding canopies and marquees may be extended to form covered walks and shelters, which should be evaluated as separate structures.

Marquees may be unengineered structures. Because of their common location at building entrances they are of concern for egress. Their evaluation and rehabilitation is a structural issue, separate from that of the main building.

Because of their locations, canopies and marquees often present a critical life safety issue. Where canopies are a horizontal extension of the structure, the structural rehabilitation must include these components and appropriate rehabilitation measures must be designed. Canopies that have been attached need careful analysis, particularly if heavy or glazed, and their attachment to the structure and bracing is critical. Permanent marquees must be rehabilitated as appropriate, depending on their design and construction characteristics.

C11.9.6.3 Acceptance Criteria

Compliance with acceptance criteria is intended to achieve the following performance:

Life Safety Performance Level. Components may not fall, and may experience only moderate displacement.

Immediate Occupancy Performance Level. Components may not fall, and shall experience only minor displacement.

C11.9.6.4 Evaluation Requirements

No commentary is provided for this section.

C11.9.7 Chimneys and Stacks

C11.9.7.1 Definition and Scope

Large chimneys and stacks are generally engineered structures, though older unreinforced brick masonry chimneys were designed and constructed using rules of thumb derived from experience. Residential brick chimneys are typically unengineered, though more recent ones may contain some reinforcing. Smaller steel and sheet metal stacks tend to be catalog items, and in seismic regions their bracing is the main concern.

C11.9.7.2 Component Behavior and Rehabilitation Concepts

The seismic evaluation of chimneys and stacks is an engineering issue, and their rehabilitation, unless there is significant material deterioration or obvious design weakness, is accomplished with bracing and improved connection to the ground or piece of equipment.

These components may fail through flexure or shear; they may fail internally, or overturn. Chimneys may disengage from a supporting wall, roof, or floor structure and cause damage to these elements.

Engineered chimneys and stacks need to be rehabilitated according to their specific design characteristics. Large masonry and, to a lesser extent, concrete chimneys may need extensive rehabilitation; a better solution may be replacement by a new steel stack, although a masonry chimney may be an integral part of the architecture and of some historic significance.

Residential chimneys can be rehabilitated by prescriptive bracing methods, though experience has shown that, unless the chimney failure causes extensive other damage to the building roof or interior, the costs of rehabilitation are similar to those of damage repair. Thus, residential chimney rehabilitation is not cost-effective unless the chimney location is such that collateral damage is likely.

C11.9.7.3 Acceptance Criteria

Compliance with acceptance criteria is intended to achieve the following performance:

Life Safety Performance Level. Chimneys and stacks located in public areas or critical to building function may not fall, but may suffer some distortion.

Immediate Occupancy Performance Level. Chimneys and stacks located in public areas or critical to building function may not fall and may suffer only minor distortion.

C11.9.7.4 Evaluation Requirements

No commentary is provided for this section.

C11.9.8 Stairs and Stair Enclosures

C11.9.8.1 Definition and Scope

No commentary is provided for this section.

C11.9.8.2 Component Behavior and Rehabilitation Concepts

When stairs are an integral part of the building structure, their evaluation should form part of the general structural evaluation. However, many stairs are prefabricated components, of steel or precast concrete, or both, which are inserted into the building structure. In these instances, if rigidly attached they may also act as structure by forming a diagonal brace between floors, creating a point of stress concentration and suffering disproportionate damage.

Stair enclosures may include a variety of separate components that can be either acceleration- or deformation-sensitive. Walls, windows, and other portions of the enclosure system may collapse into a stairwell, or stair structures may be dislodged from their supports. Safe exit may be prevented by the failure of any portion of the stair or stairwell system.

Rehabilitation may take the form of detaching the stair from the building structure at each floor, either at the top or bottom of the stair, to eliminate mutual interaction between stair and structure.

C11.9.8.3 Acceptance Criteria

Compliance with acceptance criteria is intended to achieve the following performance:

Life Safety Performance Level. Stairs may experience moderate damage but should be usable.

Immediate Occupancy Performance Level. Stairs may experience only minor damage.

C11.9.8.4 Evaluation Requirements

No commentary is provided for this section.

C11.10 Mechanical, Electrical, and Plumbing Components: Definition, Behavior, and Acceptance Criteria

C11.10.1 Mechanical Equipment

C11.10.1.1 Definition and Scope

No commentary is provided for this section.

C11.10.1.2 Component Behavior and Rehabilitation Concepts

Failure of these components consists of moving or tilting of floor- or roof-mounted equipment off its base, and deformation or loss of connection (with consequent falling) for equipment attached to vertical or horizontal structures, and failure of piping or electrical wiring connected to the equipment.

The primary object of the *Guidelines* is to ensure that the equipment remains fixed in place. The *Guidelines* do not consider the effect of shaking of the building on the internal parts of the equipment. Equipment that is suspected of being critically sensitive to this motion must be evaluated independently by the engineer, using such information as may be obtainable from the manufacturer.

It is not the intent of these *Guidelines* to require the seismic design of mechanical and electrical assemblies. When the potential for a hazard to life exists, it is expected that design efforts will focus on equipment supports, including base plates, anchorages, support lugs, legs, feet, saddles, skirts, hangers, braces, and similar items.

Many items of mechanical and electrical equipment consist of complex assemblies of mechanical and/or electrical parts that are typically manufactured using an industrial process that produces similar or identical items. Such equipment may include manufacturers' catalog items and often is designed by empirical (trial-and-error) means for functional and transportation loadings. A characteristic of such equipment is that it should be inherently rugged, in the sense that its construction and assembly provide such equipment with the ability to survive strong motions, during transportation and installation, without loss of function. By examining such equipment, an experienced design professional can usually confirm the existence of ruggedness, and can determine the need for an appropriate method and extent of specific seismic design or qualification if performance beyond the Life Safety Level is required.

It is also recognized that a number of professional and industrial organizations have developed nationally recognized codes and standards for the design and construction of specific mechanical and electrical components. In addition to providing design guidance for normal and upset operating conditions and various environmental conditions, some have developed earthquake design guidance in the context of overall mechanical or electrical design. Where continued equipment function is a matter of concern, use of such codes and standards is recommended, since their developers have familiarity with the expected failure modes of their components.

In addition, even if such codes and standards do not have earthquake design guidance, it is generally accepted that construction of mechanical and 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. Earthquake damage surveys have confirmed this.

The determination as to which equipment is subject to the *Guidelines* is based primarily on weight and location. In general, even heavy mechanical equipment does not represent a life safety threat, unless it is located where its falling or overturning might be hazardous; for example, a large unit heater suspended over an occupied area. While mechanical equipment might be made nonfunctional, this is rarely life-threatening. It might be maintained that loss of an exhaust system used as part of a fire safety strategy represents a life safety problem, but this would imply a combination of earthquake and fire, as well as trapping of occupants in a smoke-filled area for enough time for the situation to become life-threatening. If partial or complete collapse occurred, nonstructural protection would be ineffective. Though possible, this combination of events is of very low probability and the Life Safety Performance Level is defined only in terms of prevention of injury caused by direct damage. Where fully functional post-earthquake nonstructural systems are desired, higher performance must be selected.

Rehabilitation of most mechanical equipment involves a bolting and/or bracing procedure that is simple and low-cost, and generally effective in preventing often costly damage, particularly in low to moderate earthquake shaking. Thus, although rehabilitation may not be necessary from a life safety viewpoint, it may be desirable to undertake it as part of a general rehabilitation program to reduce property loss.

When the equipment is analyzed to determine seismic forces, the Default Equation can be used, because a conservative result will have little impact on the cost of the solution. Roof-mounted equipment, such as large cooling towers and packaged HVAC units, are especially vulnerable and it is recommended that the General Equation (Equations 11-2 and 11-3) be used, since this takes into account possible force amplifications due to location.

The ductility of connections, especially anchors embedded in concrete or masonry, must be evaluated with regard to the possibility for sudden brittle failure. The possibility of interaction of different pieces of equipment and structural elements as to their deformation must be considered, particularly regarding the possibility of progressive failure of a series of units.

API (1993) and AWWA (1989) provide useful discussion and information for the anchorage of equipment.

C11.10.1.3 Acceptance Criteria

Compliance with the acceptance criteria is intended to achieve the following performance:

Life Safety Performance Level. Some damage to mechanical equipment is acceptable, with the exception of overturning or falling of heavy equipment in occupied areas.

Immediate Occupancy Performance Level. Some damage is acceptable, but should be repairable without removal and replacement of major components. Equipment should not shift position.

C11.10.1.4 Evaluation Requirements

No commentary is provided for this section.

C11.10.2 Storage Vessels and Water Heaters

C11.10.2.1 Definition and Scope

This section defines fluid-containing vessels that may differ from equipment as defined in the previous section because of the reaction of the fluid within the vessel to the earthquake motions.

C11.10.2.2 Component Behavior and Rehabilitation Concepts

The failure mode for Category 1 (leg-supported) vessels will be stretching of anchor bolts, failure of legs, and consequent tilting over or possible overturning of the vessel. The failure mode for Category 2 (base-supported) vessels may be displacement off the foundation, or failure of the shell near the bottom of the tank by yielding that creates a visible bulge.

Flat bottom vessels, as described in Category 2, differ in their reaction to earthquake motions because the support of the contents is shared between the vessel itself and the direct action of the fluid on the supporting floor.

All vessels should be anchored to the building. This also applies to vessels of Category 2 in which the height-to-width ratio is low, which have often in the past been considered to be safe from failure or displacement. This is a different criterion than is used for flat bottom vessels located on the ground, where those with low height-to-diameter ratios (less than 0.50) are often considered safer if unanchored because of the beneficial effects of allowing the tank to slide to dissipate the energy of the earthquake. However, this strategy must allow for differential displacement of vessel, piping, and pipe valves, through flexible joints or bends. In addition, in buildings, the effect of the possible detrimental effect on the building by movement of a heavy load must be considered.

This section allows vessels of Category 1, which are entirely supported by the legs or skirt of the vessel and which are relatively small, to be treated the same way as the mechanical equipment of the previous section. This is because the effects of the movement of the fluid in these vessels is not as significant. The same proviso, requiring the use of the General Equation for analysis, relates to heavy items located on an upper floor of the building.

In relation to acceptable performance, failure of the tank may be acceptable—even if it leaks—if the contents can be held or diverted to either avoid a Life Safety problem or prevent damage to other components of the building. From a Damage Control viewpoint, consideration should be given to the value of the contents and the effect of spillage on the building and its contents, as well as the value of the vessel itself. For Immediate Occupancy, the main issues are the importance of the contents of the vessels to the functional operation of the building, and the restriction of damage to that which is easily repairable with minimum loss of contents. Similar to those for mechanical equipment, the Life Safety aspects of tank failure are generally not great, but rehabilitation of tanks by bracing is neither costly nor difficult, and may be very cost-beneficial in reducing property loss.

Water heaters should be restrained in accordance with prescriptive requirements that are generally available from the government jurisdiction responsible. Reference may be made to the *Memo for General Distribution No. 27* by the City of Los Angeles for such guidance. Typical general requirements for residential water heater bracing provide that the water heater should be restrained in at least two places—one near the top and one approximately one-third of the way up from the bottom—with galvanized steel straps that are at least one-half inch wide by 16 gauge. Straps should be attached into structural studs of at least 2" x 4" size or equivalent that are braced laterally by blocking and/or gypsum board or other sheathing material. Care must be taken to configure and install these braces so that they tightly restrain the tank in all horizontal directions.

Evaluation of existing tanks should include investigation of the strength of the primary elements, as well as the design of nozzles, appurtenances, platforms, ladders, and manways. Prescriptive guidelines for these elements are contained in API (1993) and AWWA (1989) in the *Guidelines*.

Evaluation should also include consideration of leakage due to corrosion and how this might be detected before it becomes a serious problem (see Appendix I in API, 1993).

C11.10.2.3 Acceptance Criteria

Compliance with the acceptance criteria is intended to achieve the following performance:

Life Safety Performance Level. Vessel remains in place without rupturing itself or its connections; minor, easily contained leakage is acceptable, unless special conditions of occupancy or tank location apply. Damage may require repair or replacement.

Immediate Occupancy Performance Level. Vessel remains in place without rupturing itself or its connections and/or vessel has positive shutoff or retention to prevent spill of contents. Damage is confined to minor repair.

C11.10.2.4 Evaluation Requirements

No commentary is provided for this section.

C11.10.3 Pressure Piping

C11.10.3.1 Definition and Scope

This section sets out an arbitrary lower limit for pressure in this piping, based on that used by most codes. This is to attempt to identify piping that has sufficient pressure to produce explosive results when rupture occurs. However, judgment should be used to identify specific piping in a given building that could produce this result.

C11.10.3.2 Component Behavior and Rehabilitation Concepts

Loss of support, causing failure at joints, is generally the mode of failure through seismic causes, which may or may not be exacerbated by the effects of corrosion. Other causes are deformation of the attached structure, or breakage from impact with adjoining materials. Piping that runs between floors or across expansion or seismic joints is drift-sensitive.

Following Project B31 in 1926, the first edition of *American Tentative Standard Code for Standard Piping* was published in 1935. Since December 1978, the ANSI B31 was reorganized as the ASME Code for Pressure Piping B31 Committee, under procedures developed by ASME and accredited by ANSI. B31 Codes, along with *ASME Boiler and Pressure Vessel Code*, Sections I through XI, are the accepted codes for these components. (See *Guidelines* references, ASME, latest edition.)

In addition to adequate support and provision for differential building movement at joints, the dynamic forces in the piping system must be evaluated along with the potential effects of corrosion on this piping. Generally, these criteria will prove more demanding than the additional effects of earthquake motions, other than possible differential movement between buildings.

Seismic rehabilitation of pressure piping focuses on adequate support and bracing, with particular attention to provision for differential movement at seismic or expansion joints. Experience has shown that most piping has sufficient inherent flexibility and ductility to accommodate building drift without damage. Thus, inserting connections to vertical piping at each floor level is neither necessary nor desirable, from the standpoint of drift-sensitive considerations, because the joint is likely to be a point of vulnerability. However, attachments or braces based on acceleration-sensitive considerations are necessary, and particular attention should be paid to large diameter heavy piping.

C11.10.3.3 Acceptance Criteria

Compliance with the acceptance criteria is intended to achieve the following performance:

Life Safety Performance Level. Minor damage may occur at some joints with some leakage but system is generally intact. Some supports may be damaged, but the system remains suspended.

Immediate Occupancy Performance Level. Minor leaks may develop at a few locations, but the system is intact.

C11.10.3.4 Evaluation Requirements

No commentary is provided for this section.

C11.10.4 Fire Suppression Piping

C11.10.4.1 Definition and Scope

This section defines piping required for fire suppression, which is treated as a separate item from other piping because of its importance and because of the large body of information that has been developed specifically for it. This section primarily applies to water sprinkler piping but also includes piping for other types of fire suppression.

C11.10.4.2 Component Behavior and Rehabilitation Concepts

Damage to this piping usually results from inadequate bracing or lack of allowance for differential movement between parts of the structure that support the piping. In addition, in recent earthquakes, sprinkler branch piping has failed because of impact with adjoining materials, typically ceiling components.

Although failure of fire suppression systems may seem an obvious instance of a Life Safety Performance Level requirement, it is not expressed as such in the *Guidelines*. For a serious life safety threat to exist, the earthquake must be accompanied by a fire that presents an immediate threat to occupants that would only be alleviated by fire sprinkler activation. Though conceivable, the probability of this combination of events is very low.

Fire sprinkler system damage, and the damage to building materials and building contents from the resulting leakage, can be extremely costly. Therefore, it is necessary that automatic, fail-safe shutoff mechanisms are in proper working order to control this potential problem. The problem of preventing water damage from sprinkler systems is particularly difficult, because a single failure in a system that may have hundreds of joints and sprinkler heads may be enough to cause extensive damage.

Observations at the 1994 Northridge earthquake (Hall, 1995), in which a number of sprinkler failures occurred, showed that failures took a number of forms. The least common, but most disruptive, was falling of pipes. A common cause of damage was incompatible motions between sprinkler piping and other ceiling or ceiling plenum components. In facilities such as garages and warehouses that lacked ceilings, failures occurred within the sprinkler system itself. These were attributed to:

- Connection deficiencies (e.g., C-clamp connections of hanger rods to beams rotated loose, or powder-driven fasteners pulled out)
- Insufficient bracing, typically in older installations
- Quality of installation work

In addition, it is possible that in some instances the building motion was too severe for even well-designed systems, properly installed according to current codes, though this point is not universally accepted. The latest (1991) *NFPA-13* edition available at the time of the earthquake (NFPA-13, 1996 in the *Guidelines*) had yet to be widely used and thus was neither validated nor invalidated by the Northridge earthquake. Some engineered sprinkler systems have more extensive bracing, not taking advantage of *NFPA-13*'s exemption of smaller branch lines.

Based on the disruptive and economic effects of sprinkler and other piping leakage in the Northridge earthquake, some suggestions have been made for the achievement of high performance in sprinkler systems, especially in essential buildings:

- Zoning systems into smaller areas, so that smaller areas can be shut off
- Using automatic or remotely controlled valves
- Requiring more rigorous training for designated personnel in immediate post-earthquake inspection and shutoff techniques

Because the requirement for sprinklers to be installed in a building is mandated from areas of the building code other than seismic, the seismic issue relates more to ensuring proper design and installation in general, rather than whether the presence of sprinklers is a Life Safety, Damage Control, or Immediate Occupancy requirement. Given that sprinklers are required, common prudence would suggest that installation issues of seismic importance be taken care of, regardless of Performance Level.

The *NFPA Fire Protection Handbook* and the *Automatic Sprinkler Systems Handbook*, both published by the National Fire Protection Association, may be used to amplify and explain that which is referenced in the *Guidelines* (NFPA, 1996). In addition, the following NFPA Standards should be used where applicable.

NFPA 11: Standard on Foam Extinguishing Systems

NFPA 12: Standard for Carbon Dioxide Extinguishing Systems

NFPA 12A, 12B: Standard for Halon Fire Extinguishing Systems

NFPA 14: Standard for the Installation of Standpipe and Hose Systems

NFPA 15: Standard for Water Spray Fixed Systems

NFPA 16: Standard for Deluge Foam-Water Sprinkler and Spray Systems

NFPA 16A: Recommended Practice for the Installation of Closed Head Foam-Water Sprinkler Systems

NFPA 17: Standard for Dry Chemical Extinguishing Systems

NFPA 17A: Standard for Wet Chemical Extinguishing Systems

C11.10.4.3 Acceptance Criteria

Compliance with the acceptance criteria is intended to achieve the following performance:

Life Safety Performance Level. Rupturing of some piping, leaving a partially functioning system. Main risers and laterals of over four inches in diameter do not fall or break. Some heads may be damaged by impact with adjoining materials, and leaks may develop at some couplings.

Immediate Occupancy Performance Level. Minor joint failures that are easily reparable; the system remains operable.

C11.10.4.4 Evaluation Requirements

No commentary is provided for this section.

C11.10.5 Fluid Piping Other than Fire Suppression

C11.10.5.1 Definition and Scope

This section separates all fluid piping that has not been covered in previous sections into hazardous and nonhazardous material conveying systems. Systems may be low-pressure or gravity.

C11.10.5.2 Component Behavior and Rehabilitation Concepts

Generally, if the piping has been recently installed to meet code requirements, secondary containment features must be in place for piping carrying hazardous materials, due to the extreme danger that accompanies failure.

The following list of possible rehabilitation measures that should be considered when evaluating a hazardous piping system and related equipment is taken from the *Piping Handbook, Sixth Edition (*Nayyar, ed., 1992).

1. Open air process units will lessen the potential for concentrating hazardous vapors.

2. Containment dikes can be added to collect spills of hazardous liquids; a diked area should be equipped with a collection sump and means for safe removal.

3. A dedicated system can be set up to collect hazardous and toxic fluid spills, to eliminate any cross-contamination with other streams.

4. The entire process area can be physically contained, with instrumentation for remote monitoring and control.

5. Ventilation can be added to remove hazardous vapor for safe disposal during emergency conditions. Ventilation may be the most important technique for controlling toxic air contaminants. General ventilation continually exchanges a supply of fresh air while exhausting air within the entire workplace. Local ventilation removes vapors, mists, and dusts continually from around equipment where hazardous fluids are contained. Either type of ventilation will require a scrubber to strip the vented air before its release to atmosphere.

   Note: The above suggestions for ventilation presuppose that electrical power continues to be available; however, under post-earthquake conditions it is likely that power will not be available, due to local or regional power outage, or failure or absence of emergency generators.

6. The inherent piping geometry, proper location of pipe anchors, pipe loops, and other integral techniques can be used to compensate for thermal expansion and contraction. To eliminate the effects of expansion and contraction, the use of mechanical devices should be avoided. Bellows and other types of expansion joints should be used only with the utmost care and adequate safeguarding.

7. Adding a pressure relief system will allow for safe discharge during upset conditions, blowdown, or cleanout. The relief system should be piped to the hazardous fluid treatment system.

8. Double-block and bleed valve arrangements can be provided on all hard piped connections where personnel may be required to enter a vessel.

9. Engineered barriers and shields at mechanical joints can protect personnel from leakage.

10. Guards or barricades can protect the piping from accidental mechanical abuse.

11. Plant arrangement should control access to hazardous areas and provide a safe distance between the hazard and the plant and/or public populated areas.

12. The system should limit the quantity of hazardous fluid that can escape in the event of a pipe rupture. Minimizing the quantity of hazardous fluid present at any time is a means of protecting people and property in the event of a piping failure.

13. Various process controls can be used to protect the system from excursions of temperature, pressure, or flow rates.

14. A systematic monitoring and leak detection program can be implemented to determine whether harmful releases are being experienced.

Seismic rehabilitation of piping focuses on adequate support, bracing, and provision for differential movement at seismic or expansion joints.

C11.10.5.3 Acceptance Criteria

Compliance with the acceptance criteria is intended to achieve the following performance:

Life Safety Performance Level. No failure of Category 1 piping within occupied areas; no leakage of contents into occupied areas.

Immediate Occupancy Performance Level. Limited damage to Category 2 piping, but system can be repaired rapidly.

C11.10.5.4 Evaluation Requirements

No commentary is provided for this section.

C11.10.6 Ductwork

C11.10.6.1 Definition and Scope

This section includes rigid air ducts, which are generally light gauge metal.

C11.10.6.2 Component Behavior and Rehabilitation Concepts

Although sheet metal ducts, especially of smaller cross sections, can tolerate large distortions and undergo small inertial loads, they have little inherent strength. They must be supported so that during a seismic event they will stay together and not rupture (where the Operational Performance Level is the goal), or not fall (where Life Safety is the goal). Joints are particularly vulnerable. Failure consists of deformation or loss of supports, leading to deformation or rupture of the ducts at joints, and permitting leakage from the system and/or malfunction of in-duct controls and devices.

In general, failure of duct systems is not a Life Safety issue. As is the case with mechanical equipment, it might be argued that loss of an exhaust system used as part of a fire safety strategy represents a Life Safety problem, but this would imply a combined earthquake and fire, and trapping of occupants in a smoke-filled area long enough for the situation to become life-threatening. Though possible, this combination of events is of very low probability. General air-handling systems can be out of action for a considerable time with no more detrimental effect than slight discomfort, depending on the intensity of occupancy and the outside climate. A Performance Level of Immediate Occupancy could, in many instances, be achieved with a nonfunctioning air handling system if temporary natural ventilation can be achieved (by opening windows and doors) and the outside climate is reasonable. In other cases (e.g., the typical hospital or data processing center), the facility cannot function without HVAC systems. A mechanical engineer should be consulted to determine the extent to which parts of a duct system may be critical for the removal of toxic substances in a laboratory, industrial plant, or other such facility.

The seismic rehabilitation of these components is relatively simple and can be designed in accordance with the Prescriptive Procedure. The designer must be aware of unusual situations where there is differential movement between different parts of the structure supporting these components, or where there are very long runs in which, during seismic motion parallel to them, large lateral forces will be generated that require larger braces than specified by the Prescriptive Procedure.

Further information regarding evaluation may be obtained from the SMACNA publications referenced in the *Guidelines.*

C11.10.6.3 Acceptance Criteria

Compliance with the acceptance criteria is intended to achieve the following performance:

Life Safety Performance Level. Ductwork systems conveying hazardous materials are not damaged; other ductwork systems may be damaged.

Immediate Occupancy Performance Level. Some damage to components but system is substantially operational, or acceptable environmental conditions can be maintained by alternative means.

C11.10.6.4 Evaluation Requirements

No commentary is provided for this section.

C11.10.7 Electrical and Communications Equipment

C11.10.7.1 Definition and Scope

No commentary is provided for this section.

C11.10.7.2 Component Behavior and Rehabilitation Concepts

The provisions for these components are very similar to those for mechanical equipment The object of the *Guidelines* is primarily to ensure ability of the equipment to remain fixed in place. The *Guidelines* do not consider the effect of shaking on the internal parts of the equipment. Equipment that is suspected of being internally sensitive to this motion must be evaluated independently by the engineer, using such information as may be obtainable from the manufacturer. Unlike much mechanical equipment, electrical and communications equipment generally does not have moving or rotating parts and is not vibration-isolated.

Failure of these components consists of moving or tilting of floor- or roof-mounted equipment off its base, deformation or loss of connection (with consequent falling) for equipment attached to vertical or horizontal structure, and failure of electrical wiring connected to the equipment.

The determination as to which equipment is subject to the *Guidelines* is based primarily on weight and location. In general, even heavy electrical and communications equipment does not represent a Life Safety threat, unless it is located where its displacement might be hazardous; a transformer suspended over an occupied area would be an example. Post-earthquake functionality issues go beyond the Life Safety Performance Level, and must be evaluated on a building-by-building basis.

Rehabilitation of most electrical and communications equipment involves prescriptive bolting and/or bracing procedures that are simple, low cost, and generally effective—particularly in low to moderate earthquakes—in preventing damage. Thus, although rehabilitation may not be necessary from a Life Safety viewpoint, it may be desirable to undertake it as part of a general rehabilitation program to reduce property loss.

The importance of each item of equipment with regard to its required Performance Level is determined by its function. Therefore, all equipment in a building must be categorized as to the effect that its failure would have on the ability of the building to satisfy criteria for the Immediate Occupancy or Operational Performance Levels.

The ductility of connections—especially large equipment anchors embedded in concrete or masonry—must be evaluated with regard to the possibility of sudden brittle failure. The possibility that different pieces of equipment and structural elements could interact, leading to their deformation, must be considered; the possible progressive failure of a series of units is a particular concern.

C11.10.7.3 Acceptance Criteria

Compliance with the acceptance criteria is intended to achieve the following performance:

Life Safety Performance Level. Some damage to equipment but heavy equipment does not detach and fall in a heavily occupied area.

Immediate Occupancy Performance Level: Some damage to components but system is substantially operational, or acceptable environmental conditions can be maintained by alternative means.

C11.10.7.4 Evaluation Requirements

No commentary is provided for this section.

C11.10.8 Electrical and Communications Distribution Components

C11.10.8.1 Definition and Scope

No commentary is provided for this section.

C11.10.8.2 Component Behavior and Rehabilitation Concepts

Electrical and communications components generally possess considerable strength or rigidity in themselves and thus need only adequate and uniform support for their protection. Supports for these distribution components are generally similar in nature to those provided for ducts, drain lines, and other small piping. The prescriptive provisions contained in the SMACNA documents referenced in the *Guidelines* are generally usable.

Failure of these components consists of failure of transmission components due to accelerations causing movement of attached equipment. Failure may also be caused by deformation or loss of supports, deformation of the attached structure, or breakage from impact with adjoining materials.

The major secondary damage caused by failure of electrical components is that of fires caused by broken power lines. This particularly applies to the residential condition where power lines are often not protected by conduit and extreme structural distortion can result in short-circuiting of power lines, resulting in fire damage to building materials or, most seriously, ignition of gas. Good general practice in the installation of power conductors is the best nonstructural safeguard against such failures.

C11.10.8.3 Acceptance Criteria

Compliance with the acceptance criteria is intended to achieve the following performance:

Life Safety Performance Level. Some damage to components but certain transmission lines required by specifics of the building design or occupancy (based on either possible fire danger or the protection of life safety systems) are protected.

Immediate Occupany Performance Level. Some damage to components that (1) are not required for life safety purposes, and (2) can be rapidly repaired; acceptable environmental and functional conditions can be maintained by alternative means.

C11.10.8.4 Evaluation Requirements

No commentary is provided for this section.

C11.10.9 Light Fixtures

C11.10.9.1 Definition and Scope

This section differentiates between light fixtures that are integral with the ceiling system, those that are surface mounted on wall or ceiling and those suspended independently.

C11.10.9.2 Component Behavior and Rehabilitation Concepts

In general, recessed and ceiling- or wall-mounted fixtures present no specific seismic problem, provided that they are securely attached to their supporting surface. To the extent that this surface may be damaged or collapse, the fixtures may be damaged and, though a rare occurrence, if heavy fixtures should fall the Life Safety consequences can be serious.

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, 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.

Fixtures supported by a ceiling grid have proven to be particularly vulnerable in recent earthquakes; their weight and hazardous design may cause injury. Such fixtures must be supported back to the structure independently of the ceiling grid. This can be easily and effectively achieved through use of backup safety wires, attached in accordance with prescriptive requirements, that are adequate at all seismic levels. Sometimes a specially designed substructure for the support of mechanical and electrical components (such as grids and trapezes) is placed between the finished ceiling and the floor or roof structure above. Bracing can generally be attached to such substructures.

Heavy chandelier fixtures should be carefully evaluated for strength of attachments, and their ability to swing safely in the event of ground motion. Suspended pendant fluorescent fixtures, often used in rows in older school rooms, have been shown to be vulnerable in recent earthquakes; these should be carefully evaluated and rehabilitated using devices that allow for movement but provide secure connections. A standard rehabilitation technique is to install backup support cables either externally—from fixture to structure above—or inside the stem.

C11.10.9.3 Acceptance Criteria

Compliance with acceptance criteria is intended to achieve the following performance:

Life Safety Performance Level
Category 1 and 2. These fixtures may be damaged, depending on damage to the ceiling or wall.

Category 3. Loss of support from the T-bar systems does not result in falling of the fixture in any occupied area.

Category 4. Fixtures do not become detached nor significantly damage any other component.

Immediate Occupancy Performance Level.
Performance is similar to that for the Life Safety Performance Level.

C11.10.9.4 Evaluation Requirements

No commentary is provided for this section.

C11.11 Furnishings and Interior Equipment: Definition, Behavior, and Acceptance Criteria

C11.11.1 Storage Racks

C11.11.1.1 Definition and Scope

Storage racks are usually steel or aluminum systems engineered to support a variety of often heavy contents loads, and may approach 20 feet in height.

C11.11.1.2 Component Behavior and Rehabilitation Concepts

In many cases these designs, while sufficient for gravity loads, may have insufficient bracing or moment-resisting capacity, or may fail by overturning or failure of foundation attachments. Racks are often improperly attached to vertical supports and have weak resistance to lateral loads.

High storage racks and their contents present a hazard that is not confined to their own failure, but includes their impact on the surrounding building structure, which can be the cause of column or even wall collapse. Storage racks, if improperly braced, often collapse or overturn in moderate or greater seismic events. Historically, these elements can be a significant safety hazard, but the principal effect of their failure is property damage and collateral loss.

Storage racks sometimes are located in areas that are essentially unoccupied, except for an occasional visit for retrieval purposes; thus the threat to life is minimal. However, the advent of very large retail discount stores, with rows of high storage racks in heavily occupied areas, represents a significant threat to life safety; the realistic analysis of seismic forces and the design of these systems need particular attention.

Even a low storage rack can, if heavily loaded, represent a significant threat if it is located in close proximity to a seated person. Rehabilitation by bracing and floor attachment should also be accompanied as much as possible by good managerial practices; this means storing heavy items toward the bottom of the racks so that falling is less likely and, if it does occur, less serious.

Storage racks can be designed to resist seismic loads through either tension-only strap bracing, bracing with compression members, or partial moment connections of the horizontal and vertical members of the rack system. The vertical loads are supported on base plates that are often not attached or inadequately attached to the floor. In the case of heavy rack systems, slab support, even if properly attached, may be inadequate to prevent failure of the slab caused by overturning loads.

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 improvement, may be required to remedy insufficient bearing or uplift load capacity.

C11.11.1.3 Acceptance Criteria

Compliance with the acceptance criteria is intended to achieve the following performance:

Life Safety Performance Level. In Life Safety-critical locations with occupancy in close proximity, no upset of racks in excess of four feet in height; some damage to the rack system itself.

Immediate Occupancy Performance Level. No upset of racks or collateral damage to supporting structure but minor damage to rack system.

C11.11.1.4 Evaluation Requirements

No commentary is provided for this section.

C11.11.2 Bookcases

C11.11.2.1 Definition and Scope

Unlike storage racks, bookcases are usually under ten feet in height, but they often exist in areas—such as libraries—with high human occupancy, where their failure could result in injury or loss of life.

C11.11.2.2 Component Behavior and Rehabilitation Concepts

Bookcases may be heavily stacked and in close proximity to a seated person; even a low bookcase represents a significant threat. Bookcases are usually not engineered and, while sufficiently strong to support gravity loads, they may be inadequate to transfer lateral loads internally, and they are often inadequately attached to the supporting floors or adjacent columns, walls, or other structural members. The historic behavior of bookcases includes numerous instances of overturning failure. There have also been significant cases—in library installations of large fully loaded bookcases—of internal racking or buckling failure, usually along the longitudinal axis.

Engineering solutions for rehabilitation usually require a systematic Analytical Procedure. Options often include improvements to the longitudinal lateral stability of the bookcase by the addition of strap cross-bracing or panelized stiffening, using plywood or other materials, along with attachments to the supporting floor structure. Another common rehabilitation technique involves improving attachments to the supporting floor structure and connecting the top of the bookcases, through a series of struts, horizontally to each other and to adjacent supporting wall or column structure. This technique reduces overturning forces.

C11.11.2.3 Acceptance Criteria

Compliance with the acceptance criteria is intended to achieve the following performance:

Life Safety Performance Level. No upset of bookcases in excess of four feet in height in occupied areas. Some damage to the system. Most volumes restrained on the shelves.

Immediate Occupancy Performance Level. No upset of bookcases or collateral damage to supporting structure. Minor damage to system.

C11.11.2.4 Evaluation Requirements

No commentary is provided for this section.

C11.11.3 Computer Access Floors

C11.11.3.1 Definition and Scope

Computer access floors are available in a variety of types, but are usually made up of two basic components. The first is a system of supporting legs or stanchions and horizontal beams, laid out to accommodate the second part of the system, an access floor panel. Supporting structures are usually designed and constructed of steel, while the floor panels can be of wood, metal, concrete, or composite construction. Access floors are designed for various and often changing arrangements. They are generally well engineered for the support of vertical loads.

C11.11.3.2 Component Behavior and Rehabilitation Concepts

Access floors rarely fail in earthquakes, but because they carry the lateral loads developed by the mass of computer or other electronic systems that they support, failure does sometimes occur, by either dislodgment of the panels, failure of the supporting stanchions and horizontal members, or both. In many of these cases, base plates of the stanchions are inadequately connected, or not connected at all, to the supporting floor system.

The implications of poor seismic performance in access floors are not usually related to Life Safety so much as to business recovery, since the equipment they support is often important to communications or data processing. Rehabilitation of access floors usually includes (1) improving attachment of computer and communication racks through the access floor panels to the supporting steel structure or to the underlying floor system, and (2) improving the lateral-load-carrying capacity of the steel stanchion system by installing braces, improving the connection of stanchion base to the supporting floor, or both.

A useful discussion of all aspects of the protection of data processing equipment will be found in Olson (1987).

C11.11.3.3 Acceptance Criteria

Compliance with acceptance criteria is intended to achieve the following performance:

Life Safety Performance Level. Not applicable.

Immediate Occupancy Performance Level. No failure occurs; only minor displacement of supporting structure occurs. Some displacement of panels occurs.

C11.11.3.4 Evaluation Requirements

No commentary is provided for this section.

C11.11.4 Hazardous Materials Storage

C11.11.4.1 Definition and Scope

In this document, the scope is limited to engineering techniques for protecting permanently installed containers. Propane gas tanks and their supporting legs are included, while containers for hazardous materials stored on counter tops, shelves, or desktops are typically excluded due to the large variation in conditions, although these hazards may be significant. See FEMA 74 (FEMA, 1994).

C11.11.4.2 Component Behavior and Rehabilitation Concepts

The containers that hold hazardous materials are generally not engineered, with the exception of large chemical containers or gas cylinders. In many cases, the supports for even heavy tanks have not been adequately designed to resist lateral loading. The historic performance of these elements includes numerous instances of broken glass containers thrown from shelves and counter tops, as well as tanks dislodged from their supports.

These components usually fail by sliding or overturning, and break only on impact. An additional concern is the potential for rupture of connecting piping and tubing. Rehabilitation measures are usually prescriptive; solutions run from the installation of wire or transparent plastic barriers—to prevent shelf-stored hazardous materials and containers from falling—to improvements in lateral bracing and foundation attachment for heavy tanks.

C11.11.4.3 Acceptance Criteria

Compliance with the acceptance criteria is intended to achieve the following performance:

Life Safety Performance Level. No displacement, breakage, or disconnection of a container in close proximity to occupancy where leakage can cause immediate life threat.

Immediate Occupancy Performance Level. No displacement, breakage, or disconnection of a container in a functional critical area that allows a release of materials individually or collectively hazardous. Minor damage in other areas.

C11.11.4.4 Evaluation Requirements

No commentary is provided for this section.

C11.11.5 Computer and Communication Racks

C11.11.5.1 Definition and Scope

The rack systems included in this section are similar in construction to storage racks discussed in Section 11.11.1. They typically support expensive and sensitive electronic equipment, including computers, network servers, and telecommunications equipment. The equipment itself is not included in the definition, although functional and property losses may result from their failure.

C11.11.5.2 Component Behavior and Rehabilitation Concepts

Computer and communication racks are usually designed to adequately support the vertical loads of the equipment that they contain; in some cases, they are integral with that equipment and form the outer computer compartment. Historic performance includes overturning failure due to inadequate attachment to supporting access or structural floor systems, as well as racking (particularly longitudinal) associated with inadequate bracing or shear panels. Because the systems are often supported on computer access floors, a combination of measures, including both elements, may need to be implemented in order to assure adequate rack performance.

Rehabilitation measures typically require an Analytical Procedure, including the estimated weight of the rack contents, to establish forces on the components. Rehabilitation often includes bracing or additional panels within the rack itself, as well as improvements to the attachment of the rack base through the access floor panel to the supporting structure. Positive connections of equipment to rack are also frequently needed.

C11.11.5.3 Acceptance Criteria

Compliance with acceptance criteria is intended to achieve the following performance:

Life Safety Performance Level. Not applicable.

Immediate Occupancy Performance Level. No upset of racks or collateral damage to supporting structure. Minor damage and/or distortion of racks. Distortion does not disengage electronic connectors or damage equipment.

C11.11.5.4 Evaluation Requirements

No commentary is provided for this section.

C11.11.6 Elevators

C11.11.6.1 Definition and Scope

The definition of elevators in this sections is intended to encompass the entirety of elevator machinery, shafts, cars, and supporting rooms.

C11.11.6.2 Component Behavior and Rehabilitation Concepts

Rehabilitation of elevators is typically aimed at safety rather than immediate post-earthquake operation; their use for immediate escape is not contemplated. Current seismic provisions for elevators are aimed at safe shutdown rather than continued functionality. After even moderate shaking, the majority of elevators will need inspection after shutdown before they can be regarded as safe. This fact alone means that elevators cannot be regarded as available for escape or rescue.

Many parts of elevator systems—typically, the supporting frames and members—are engineered systems, but some are not, and those that are engineered may not have been designed with seismic loads in mind. Engineered systems will have been designed for safety in ordinary operation; those of more modern construction may also include restraints or other devices that improve seismic performance. Shaft walls and the construction of machinery room walls are often not engineered and must be considered in a similar way as 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. Shaft walls can be damaged in the same way as other partitions, and materials may fall down the shaft onto the cab. 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.

C11.11.6.3 Acceptance Criteria

Compliance with acceptance criteria is intended to achieve the following performance:

Life Safety Performance Level. Elevators may be out of service, but counterweights are not dislodged.

Immediate Occupancy Performance Level. Minor damage occurs, but the elevators, shafts, and necessary equipment are functional. Elevators are capable of operating when power is available.

C11.11.6.4 Evaluation Requirements

No commentary is provided for this section.

C11.11.7 Conveyors

C11.11.7.1 Definition and Scope

Conveyors include the belts, supporting trusses, and machinery in material conveyors used to move merchandise, luggage, packages, or other products. The equipment is often complex and includes many pieces of equipment similar to those described in other sections of the *Guidelines.*

C11.11.7.2 Component Behavior and Rehabilitation Concepts

These systems are often both acceleration- and deformation-sensitive, and experience shows that seismic events can dislodge or deform individual pieces of the system in a manner similar to the effects on other heavy mechanical equipment.

Conveyors are engineered systems, but many are not designed with seismic loads in mind. They have been designed for ordinary operating loads; those of more modern construction may also include anchorage, restraints, or other devices that improve seismic performance. Rehabilitation measures include a variety of techniques taken from specific component sections for mechanical equipment. Rehabilitation of supporting trusses or other structures may include bracing and additional strength where necessary, based on the requirements of Chapter 5.

C11.11.7.3 Acceptance Criteria

Compliance with acceptance criteria is intended to achieve the following performance:

Life Safety Performance Level. Not applicable.

Immediate Occupancy Performance Level. Minor damage occurs, but conveyors and equipment are operable.

C11.11.7.4 Evaluation Requirements

No commentary is provided for this section.

C11.12 Definitions

No commentary is provided for this section.

C11.13 Symbols

No commentary is provided for this section.

C11.14 References

AAMA, 1996a, *Aluminum Design Guide Curtain Wall Manual*, American Architectural Manufacturers Association, Schaumburg, Illinois.

AAMA, 1996b, *Rain Screen Principle and Pressure Equalized Wall Design*, American Architectural Manufacturers Association, Schaumburg, Illinois.

Anco, 1983, *Seismic Hazard Assessment of Nonstructural Ceiling Components - Phase 1*, Anco Engineers Inc., Culver City, California.

API, 1993, *Welded Steel Tanks for Oil Storage*, API STD 650, American Petroleum Institute, Washington, D.C.

ASHRAE, 1991, "Seismic Restraint Design," Chapter 49, *ASHRAE Handbook*, American Society of Heating, Refrigerating and Air-Conditioning Engineers, Inc., Atlanta, Georgia.

ASTM, 1994, *Science and Technology of Building Seals, Sealants, Glazing and Waterproofing: Third Volume*, STP1254, American Society for Testing and Materials, Conshohocken, Pennsylvania.

ASTM, 1995, *Manual 21/Modern Stone Cladding; Design and Installation of Exterior Dimension Stone Systems*, American Society for Testing and Materials, Conshohocken, Pennsylvania.

ATC, 1978, *Tentative Provisions for the Development of Seismic Regulations for Buildings*, Report No. ATC-3-06, Applied Technology Council, Redwood City, California.

ATC, 1992, *Seismic Design and Performance of Equipment and Nonstructural Elements in Building and Industrial Structures*, Report No. ATC-29, Applied Technology Council, Redwood City, California.

AWWA, 1989, *Welded Steel Tanks for Water Storage*, D100, American Water Works Association, Denver, Colorado.

Ayres, J. M., 1993, "History of Earthquake Resistive Design Building Mechanical Systems," *ASHRAE Transactions: Symposia CH-93-1-1*, American Society of Heating, Refrigerating and Air-Conditioning Engineers, Inc., Atlanta, Georgia.

Ayres, J. M., and Sun, T. Y., 1973a, "Nonstructural Damage to Buildings," *The Great Alaska Earthquake of 1964*, Engineering, National Academy of Sciences, Washington, D.C.

Ayres, J. M., and Sun, T. Y., 1973b, "Nonstructural Damage," *The San Fernando, California, Earthquake of February 9, 1971*, Vol. 1B, National Oceanic and Atmospheric Administration, Washington, D.C.

BSSC, 1992a, *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, 1992b, *NEHRP Handbook for the Seismic Evaluation of Existing Buildings*, developed by the Building Seismic Safety Council for the Federal Emergency Management Agency (Report No. FEMA 178), 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.

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.

CSSC, 1995, *Performance of Nonstructural Elements, Background Report B19*, Governor's Executive Order W-78-94, California Seismic Safety Commission, Sacramento, California.

DOE, 1995, *Seismic Safety Manual, A Practical Guide for Facility Managers and Earthquake Engineers*, Department of Energy, Washington, D.C.

Drake, R. M., and Bachman, R. E., 1995, "Interpretation of Instrumental Building Seismic Data and Implications for Building Codes," *Proceedings*, SEAOC Annual Convention, Structural Engineers Association of California, Sacramento, California.

FEMA, 1994, *Reducing the Risks of Nonstructural Earthquake Damage: A Practical Guide*, Report No. FEMA 74, Federal Emergency Management Agency, Washington, D.C.

Hall, J. F., ed., 1995, "Nonstructural Damages," *Northridge Earthquake Reconnaissance Report, Vol. 1, Earthquake Spectra*, Supplement C to Volume 11, Earthquake Engineering Research Institute, Oakland, California.

Holmes, W., and Reitherman, R. K., 1994, Testimony before the California Seismic Safety Commission, Special Hearing on the 1994 Northridge, California Earthquake, on Nonstructural Damage (10 February), and on Hospitals (11 February), Van Nuys, California.

ICBO, 1994, *Uniform Building Code*, International Conference of Building Officials, Whittier, California.

MIA, 1994, *Masonry Veneer*, Masonry Institute of America, Los Angeles, California.

Nayyar, M. L., ed., 1992, *Piping Handbook, Sixth Edition*, McGraw-Hill, Inc., New York, New York.

NFPA, 1996, *Standard for the Installation of Sprinkler Systems*, NFPA-13, National Fire Protection Association, Quincy, Massachusetts.

Olson, R. A., 1987, *Data Processing Facilities: Guidelines for Earthquake Mitigation*, VSP Inc., Sacramento, California.

PCA, 1995, *Portland and Cement Plaster Stucco Manual*, Portland Cement Association, Chicago, Illinois.

RMS, 1995, *Development of a Standardized Earthquake Loss Estimation Methodology, Volume 1*, Prepared by Risk Management Solutions Inc., for the National Institute of Building Sciences, Washington, D.C.

Sheet Metal Industry Fund of Los Angeles, 1976, *Guidelines for Seismic Restraints of Mechanical Systems*, Los Angeles, California.

SMACNA, 1980, *Rectangular Industrial Duct Construction Standards*, Sheet Metal and Air Conditioning Contractors National Association Inc., Chantilly, Virginia.

SMACNA, 1982, *Guidelines for Seismic Restraint of Mechanical Systems and Plumbing Piping Systems*, Sheet Metal Industry Fund of Los Angeles and Plumbing and Piping Industry Council, Sheet Metal and Air Conditioning Contractors National Association, Chantilly, Virginia.

SMACNA, 1991, *Seismic Restraint Guidelines for Mechanical Equipment*, Sheet Metal and Air Conditioning Contractors National Association, Chantilly, Virginia.

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.

B. Seismic Rehabilitation Guidelines Project Participants

Project Oversight Committee

Chairman

Eugene Zeller
Director of Planning and Building
Department of Planning and Building
Long Beach, California

ASCE Members

Paul Seaburg
Office of the Associate Dean
College of Engineering and Technology
Omaha, Nebraska

Ashvin Shah
Director of Engineering
American Society of Civil Engineers
Washington, D.C.

ATC Members

Thomas G. Atkinson
Atkinson, Johnson and Spurrier
San Diego, California

Christopher Rojahn
Executive Director
Applied Technology Council
Redwood City, California

BSSC Members

Gerald H. Jones
Consultant
Kansas City, Missouri

James R. Smith
Executive Director
Building Seismic Safety Council
Washington, D.C.

ASCE Project Participants

Project Steering Committee

Vitelmo V. Bertero
Earthquake Engineering Research Center
University of California
Richmond, California

William J. Hall
University of Illinois at Urbana-Champaign
Urbana, Illinois

Clarkson W. Pinkham
S. B. Barnes Associates
Los Angeles, California

Paul Seaburg
University of Omaha
Omaha, Nebreska

Roland L. Sharpe
Consulting Structural Engineer
Los Altos, California

Jon S. Traw
International Council of Building Officials
Whittier, California

Users Workshops

Tom McLane, Manager
American Socity of Civil Engineers
Washington, D.C.

Debbie Smith, Coordinator
American Socity of Civil Engineers
Washington, D.C.

Research Synthesis Subcontractor

James O. Jirsa
University of Texas
Austin, Texas

Special Issues Subcontractor

Melvyn Green
Melvyn Green and Associates
Torrance, California

ATC Project Participants

Senior Technical Comettee

Principal Investigator (PI)

Christopher Rojahn
Applied Technology Council
Redwood City, California

Co-PI and Project Director

Daniel Shapiro
SOH & Associates
San Francisco, California

Co-Project Director

Lawrence D. Reaveley
University of Utah
Salt Lake City, Utah

Senior Technical Advisor

William T. Holmes
Rutherford & Chekene
San Francisco, California

Technical Advisor

Jack Moehle
EERC, UC Berkeley
Richmond, California

ATC Board Representative (Ex-Officio)

Tom Atkinson
Atkinson, Johnson, & Spurrier
San Diego, California

General Requirements Team

Ronald O. Hamburger, Team Leader
EQE International
San Francisco, California

Sigmund A. Freeman
Wiss, Janney, Elstner Associates
Emeryville, California

Prof. Peter Gergely (deceased)
Cornell University
Ithaca, New York

Richard A. Parmelee
Consulting Engineer
St. George, Utah

Allan R. Porush
Dames & Moore
Los Angeles, California

Modeling and Analysis Team

Mike Mehrain, Team Leader
Dames & Moore
Los Angeles, California

Ronald P. Gallagher
R. P. Gallagher Associates
San Francisco, California

Helmut Krawinkler
Stanford University
Stanford, California

Guy J. P. Nordenson
Ove Arup & Partners
New York, New York

Maurice S. Power
Geomatrix Consultants, Inc.
San Francisco, California

Andrew S. Whittaker
EERC, UC Berkeley
Richmond, California

Geotechnical/Foundations Team

Jeffrey R. Keaton, Team Leader
AGRA Earth & Environmental
Phoenix, Arizona

Craig D. Comartin
Consulting Engineer
Stockton, California

Paul W. Grant
Shannon & Wilson, Inc.
Seattle, Washington

Geoffrey R. Martin
University of Southern California
Los Angeles, California

Maurice S. Power
Geomatrix Consultants, Inc.
San Francisco, California

Concrete Team

Jack P. Moehle, Co-Team Leader
EERC, UC Berkeley
Richmond, California

Lawrence D. Reaveley, Co-Team Leader
University of Utah
Salt Lake City, Utah

James E. Carpenter
Bruce C. Olsen Consulting Engineers
Seattle, Washington

Jacob Grossman
Rosenwasser/Grossman Cons Engrs.
New York, New York

Paul A. Murray
Stanley D. Lindsey & Associates
Nashville, Tennessee

Joseph P. Nicoletti
URS/John A. Blume and Associates
San Francisco, California

Kent B. Soelberg
Rutherford & Chekene
Boise, Idaho

James K. Wight
University of Michigan
Ann Arbor, Michigan

Masonry Team

Daniel P. Abrams, Team Leader
University of Illinois
Urbana, Illinois

Samy A. Adham
Agbabian Associates
Pasadena, California

Gregory R. Kingsley
KLFA of Colorado
Golden, Colorado

Onder Kustu
Oak Engineering
Belmont, California

John C. Theiss
EQE - Theiss
St. Louis, Missouri

Steel Team

Douglas A. Foutch, Team Leader
University of Illinois
Urbana, Illinois

Navin R. Amin
Skidmore, Owings & Merrill
San Francisco, California

James O. Malley
Degenkolb Engineers
San Francisco, California

Charles W. Roeder
University of Washington
Seattle, Washington

Thomas Z. Scarangello
Thornton-Tomasetti
New York, New York

Wood Team

John M. Coil, Team Leader
Coil & Welsh
Tustin, California

Jeffery T. Miller
Reaveley Engineers & Assoc., Inc.
Salt Lake City, Utah

Robin Shepherd
Forensic Expert Advisers, Inc.
Santa Ana, California

William B. Vaughn
Vaughn Engineering
Lafayette, California

New Technologies Team

Charles A. Kircher, Team Leader
Charles Kircher & Associates
Mountain View, California

Michael C. Constantinou
State University of New York at Buffalo
Buffalo, New York

Andrew S. Whittaker
EERC, UC Berkeley
Richmond, California

Nonstructural Team

Christopher Arnold, Team Leader
Building Systems Development
Palo Alto, California

Richard L. Hess
Hess Engineering, Inc.
Los Alamitos, California

Frank E. McClure
Consulting Structural Engineer
Orinda, California

Todd W. Perbix
RSP/EQE
Seattle, Washington

Simplified Rehabilitation Team

Chris D. Poland, Team Leader
Degenkolb Engineers
San Francisco, California

Leo E. Argiris
Ove Arup & Partners
New York, New York

Thomas F. Heausler
Heausler Structural Engineers
Kansas City, Missouri

Evan Reis
Degenkolb Engineers
San Francisco, California

Tony Tschanz
Skilling Ward Magnusson
Barkshire, Inc.
Seattle, Washington

Qualification of In-Place Materials Consultants

Charles J. Hookham (Lead)
Black & Veatch
Ann Arbor, Michigan

Dr. Richard Atkinson (deceased)
Atkinson-Noland & Associates
Boulder, Colorado

Ross Esfandiari
Ross Engineering Services
Walnut Creek, California

Language & Format Consultant

James R. Harris
J. R. Harris & Company
Denver, Colorado

Report Preparation Consultants

Roger E. Scholl (deceased), Lead
CounterQuake Corporation
Redwood City, California

Robert K. Reitherman
The Reitherman Company
Half Moon Bay, California

Document Production

Lasselle-Ramsay, Inc.
Mountain View, California

ATC Staff

Christopher Rojohn
Executive Director

A. Gerald Brady
Deputy Executive Director

Patty Christofferson
Manager, Administration & Public Relations

Peter N. Mork
Computer Specialist

BSSC Project Participants

1997 Board of Direction

Chairman

Eugene Zeller
Director of Planning and Building
Department of Planning and Building
Long Beach, California

Vice Chairman

William W. Stewart, FAIA
Stewart•Schaberg/Architects
Clayton, Missouri
(representing the American Institute of Architects)

Secretary

Mark B. Hogan
Vice President of Engineering
National Concrete Masonry Association
Herndon, Virginia

Members

James E. Beavers, Ex officio
James E. Beavers Consultants
Oak Ridge, Tennessee

Eugene Cole
Cole, Yee, Schubert and Associates
Carmichael, California
(representing the Structural Engineers Association of California)

S. K. Ghosh
Portland Cement Association
Skokie, Illinois

Nestor Iwankiw
Vice President, Technology and Research
American Institute of Steel Construction
Chicago, Illinois

Gerald H. Jones
Kansas City, Missouri
(representing the National Institute of Building Sciences)

Joseph Nicoletti
Senior Consultant
URS/John A. Blume & Associates
San Francisco, California
(representing the Earthquake Engineering Research Institute)

John R. "Jack" Prosek
Project Manager
Turner Construction Company
San Francisco, California
(representing Associated General Contractors of America)

W. Lee Shoemaker, Ph.D.
Director of Research and Engineering
Metal Building Manufacturers Association
Cleveland, Ohio

John C. Theiss
Vice President
EQE - Theiss
St. Louis, Missouri
(representing the American Society of Civil Engineers)

Charles H. Thornton
Chairman/Principal
Thornton-Tomasetti Engineers
New York, New York
(representing the Applied Technology Council)

David P. Tyree
Regional Manager
American Forest and Paper Association
Colorado Springs, Colorado

Members (continued)

David Wismer
Director of Planning and Code Development
Department of Licenses and Inspections
Philadelphia, Pennsylvania
(representing Building Officials and Code Administrators International)

Richard Wright
Director
Building and Fire Research Laboratory
National Institute of Standards and Technology
Gaithersburg, Maryland

BSSC Staff

James R. Smith
Executive Director

Thomas R. Hollenbach
Deputy Executive Director

Larry Anderson
Director, Special Projects

Claret M. Heider
Technical Writer-Editor

Mary Marshall
Administrative Assistant

Societal Issues Subcontractor

Robert A. Olson
Robert Olson Associates, Inc.
Sacramento, California

BSSC Project Committee

Chairman

Warner Howe
Consulting Structural Engineer
Germantown, Tennessee

Consultant

Robert A. Olson
Robert Olson Associates, Inc.
Sacramento, California

Members

Gerald H. Jones
Kansas City, Missouri

Harry W. Martin
American Iron and Steel Institute
Auburn, California

Allan R. Porush
Structural Engineer
Dames and Moore
Los Angeles, California

F. Robert Preece
Preece/Goudie and Associates
San Francisco, California

William W. Stewart, FAIA
Stewart Schaberg/Architects
Clayton, Missouri

Seismic Rehabilitation Advisory Panel

Chairman

Gerald H. Jones
Kansas City, Missouri

Members

David E. Allen
Structures Division
Institute of Research in Construction
National Research Council of Canada
Ottawa, Ontario, Canada

John Battles
Southern Building Code Congress, International
Birmingham, Alabama

David C. Breiholz
Chairman, Existing Buildings Committee
Structural Engineers Association of California
Lomita, California

Michael Caldwell
American Institute of Timber Construction
Englewood, Colorado

Gregory L. F. Chiu
Institute for Business and Home Safety
Boston, Massachusetts

Terry Dooley
Morley Construction Company
Santa Monica, California

Susan M. Dowty
International Conference of Building Officials
Whittier, California

Steven J. Eder
EQE Engineering Consultants
San Francisco, California

S. K. Ghosh
Portland Cement Association
Skokie, Illinois

Barry J. Goodno
Professor
School of Civil Engineering
Georgia Institute of Technology
Atlanta, Georgia

Charles C. Gutberlet
U.S. Army Corps of Engineers
Washington, D.C.

Warner Howe
Consulting Structural Engineer
Germantown, Tennessee

Howard Kunreuther
Wharton School
University of Pennsylvania
Philadelphia, Pennsylvania

Harry W. Martin
American Iron and Steel Institute
Auburn, California

Robert McCluer
Building Officials and Code Administrators, International
Country Club Hills, Illinois

Margaret Pepin-Donat
National Park Service Retired
Edmonds, Washington

William Petak
Professor, Institute of Safety and Systems Management
University of Southern California
Los Angeles, California

Howard Simpson
Simpson, Gumpertz and Heger
Arlington, Massachusetts

William W. Stewart, FAIA
Stewart Schaberg/Architects
Clayton, Missouri

James E. Thomas
Duke Power Company
Charlotte, North Carolina

L. Thomas Tobin
Tobin and Associates
Mill Valley, California

Representatives of BSSC Member Organizations and their Alternates
(as of September 1997)

AFL-CIO Building and Construction Trades Department

Representative

Sandra Tillett
AFL-CIO Building and Construction Trades
Washington, D.C.

Alternate

Pete Stafford
Center to Protect Workers' Rights
Washington, D.C.

AISC Marketing, Inc.

Representative

Robert Pyle
AISC Marketing, Inc.
Buena Park, California

American Concrete Institute

Representative

Arthur J. Mullkoff
American Concrete Institute
Farmington Hills, Michigan

Alternate

Ward R. Malisch
American Concrete Institute
Farmington Hills, Michigan

American Consulting Engineers Council

Representative

Roy G. Johnston
Brandow and Johnston Associates
Los Angeles, California

Alternate

Edward Bajer
American Consulting Engineers Council
Washington, D.C.

American Forest and Paper Association

Representative

David P. Tyree, P.E.
American Forest and Paper Association
Colorado Springs, Colorado

Alternate

Bradford K. Douglas, P.E.
American Forest and Paper Association
Washington, D.C.

American Institute of Architects

Representative

William W. Stewart, FAIA
Stewart-Schaberg/Architects
Clayton, Missouri

Alternate

Gabor Lorant
Gabor Lorant Architect Inc.
Phoenix, Arizona

American Institute of Steel Construction

Representative

Nestor Iwankiw
American Institute of Steel Construction
Chicago, Illinois

American Insurance Services Group, Inc.

Representative

John A. Mineo
American Insurance Services Group, Inc.
New York, New York

Alternate

Phillip Olmstead
ITT Hartford Insurance Group
Hartford, Connecticut

American Iron and Steel Institute

Representative

Harry W. Martin
American Iron and Steel Institute
Auburn, California

American Plywood Association

Representative

Kenneth R. Andreason
American Plywood Association
Tacoma, Washington

Alternate

William A. Baker
American Plywood Association
Tacoma, Washington

American Society of Civil Engineers

Representative

John C. Theiss
EQE - Theiss
St. Louis, Missouri

Alternate

Ashvin Shah
Scarsdale, New York

American Society of Civil Engineers - Kansas City Chapter

Representative

Harold Sprague
Black & Veatch
Overland, Missouri

Alternate

Brad Vaughan
Black & Veatch
Overland, Missouri

American Society of Heating, Refrigerating and Air-Conditioning Engineers, Inc.

Representative

William Staehlin
State of California
Sacramento, California

Alternate

Bruce D. Hunn, Ph.D.
ASHRE
Atlanta, Georgia

American Society of Mechanical Engineers

Representative

Evangelos Michalopoulos
Hartford Steam Boiler Inspection and Insurance Company
Hartford, Connecticut

Alternate

Ronald W. Haupt
Pressure Piping Engineering Associates
Foster City, California

American Welding Society

Representative

Hardy C. Campbell III
American Welding Society
Miami, Florida

Alternate

Charles R. Fassinger
American Welding Society
Miami, Florida

Applied Technology Council

Representative

Christopher Rojahn
Applied Technology Council
Redwood City, California

Alternate

Charles N. Thornton
Thornton-Tomasetti
New York, New York

Associated General Contractors of America

Representative

Jack Prosek
Turner / VANIR Construction Management
Oakland, California

Alternate

Christopher Monek
Associated General Contractors of America
Washington, D.C.

Association of Engineering Geologists

Representative

Ellis Krinitzsky
Army Corps of Engineers
Vicksburg, Mississippi

Alternate

Patrick J. Barosh
Patrick J. Barosh & Associates
Concord, Massachusetts

Association of Major City Building Officials

Representative

Arthur J. Johnson, Jr.
City of Los Angeles
Department of Building and Safety
Los Angeles, California

Alternate

Karl Deppe
City of Los Angeles
Department of Building and Safety
Los Angeles, California

Brick Institute of America

Representative

J. Gregg Borchelt
Brick Institute of America
Reston, Virginia

Alternate

Mark Nunn
Brick Institute of America
Reston, Virginia

Building Officials and Code Administrators International

Representative

David Wismer
Department of Licenses and Inspections
Philadelphia, Pennsylvania

Alternate

Paul K. Heilstedt
BOCA, International
Country Club Hills, Illinois

Building Owners and Managers Association International

Representative

Michael Jawer
BOMA, International
Washington, D.C.

California Geotechnical Engineers Association

Representative

Alan Kropp
Alan Kropp & Associates
Berkeley, California

Alternate

John A. Baker
Anderson Geotechnical Consultants
Roseville, California

California Seismic Safety Commission

Representative

Fred Turner
Seismic Safety Commission
Sacramento, California

Canadian National Committee on Earthquake Engineering

Representative

R. H. Devall
Read Jones Christoffersen Ltd.
Vancouver, British Columbia, Canada

Alternate

D. A. Lutes
National Research Council of Canada
Division of Research Building
Ottawa, Ontario, Canada

Concrete Masonry Association of California and Nevada

Representative

Stuart R. Beavers
Concrete Masonry Association of California and Nevada
Citrus Heights, California

Alternate

Daniel Shapiro
SOH & Associates, Structural Engineers
San Francisco, California

Concrete Reinforcing Steel Institute

Representative

David P. Gustafson
Concrete Reinforcing Steel Institute
Schaumburg, Illinois

Alternate

H. James Nevin
Concrete Reinforcing Steel Institute
Glendora, California

Division of the State Architect

Representative

Vilas Mujumdar
Division of the State Architect
Sacramento, California

Alternate

Alan Williams
Division of the State Architect
Sacramento, California

Earthquake Engineering Research Institute

Representative

Joseph Nicoletti
URS Consultants
San Francisco, California

Alternate

F. Robert Preece
Preece/Goudie & Associates
San Francisco, California

Hawaii State Earthquake Advisory Board

Representative

Paul Okubo, Ph. D.
Hawaii State Earthquake Advisory Board
Department of Defense
Honolulu, Hawaii

Alternate

Gary Chock
Martin, Bravo & Chock, Inc.
Honolulu, Hawaii

Insulating Concrete Form Association

Representative

Dick Whitaker
Insulating Concrete Form Association
Glenview, Illinois

Alternate

Dan Mistick
Portland Cement Association
Skokie, Illinois

Institute for Business and Home Safety

Representative

Gregory L.F. Chiu
Institute for Business and Home Safety
Boston, Massachusetts

Alternate

Karen Gahagan
Institute for Business and Home Safety
Boston, Massachusetts

Interagency Committee on Seismic Safety in Construction

Representative

Richard Wright
National Institute of Standards & Technology
Gaithersburg, Maryland

Alternate

H. S. Lew
National Institute of Standards and Technology
Gaithersburg, Maryland

International Conference of Building Officials

Representative

Rick Okawa
International Conference of Building Officials
Whittier, California

Alternate

Susan M. Dowty
International Conference of Building Officials
Laguna Niguel, California

International Masonry Institute

Representative

Richard Filloramo
International Masonry Institute
Glastonbury, Connecticut

Alternate

Diane Throop
International Masonry Instittute
Great Lakes Office
Ann Arbor, Michigan

Masonry Institute of America

Representative

John Chrysler
Masonry Institute of America
Los Angeles, California

Alternate

James E. Amrhein
Masonry Institute of America
Los Angeles, California

Metal Building Manufacturers Association

Representative

W. Lee Shoemaker, Ph.D., P.E.
Metal Building Manufacturers Association
Cleveland, Ohio

Alternate

Joe N. Nunnery
AMCA Building Division
Memphis, Tennessee

National Association of Home Builders

Representative

Ed Sutton
National Association of Home Builders
Washington, D.C.

National Concrete Masonry Association

Representative

Mark B. Hogan, P.E.
National Concrete Masonry Association
Herndon, Virginia

Alternate

Phillip J. Samblanet
National Concrete Masonry Association
Herndon, Virginia

National Conference of States on Building Codes and Standards

Representative

Richard T. Conrad, AIA
State Historical Building Safety Board
Sacramento, California

Alternate

Robert C. Wible
National Conference of States on Building Codes and Standards
Herndon, Virginia

National Council of Structural Engineers Associations

Representative

Howard Simpson
Simpson, Gumpertz & Heger
Arlington, Massachusetts

Alternate

W. Gene Corley
Construction Technology Laboratories
Skokie, Illinois

National Elevator Industry, Inc.

Representative

George A. Kappenhagen
Schindler Elevator Corporation
Morristown, New Jersey

National Fire Sprinkler Association, Inc.

Representative

Russell P. Fleming
National Fire Sprinkler Association
Patterson, New York

Alternate

Kenneth E. Isman
National Fire Sprinkler Association
Patterson, New York

National Institute of Building Sciences

Representative

Gerald H. Jones
Kansas City, Missouri

National Ready Mixed Concrete Association

Representative

Anne M. Ellis, P.E.
National Ready Mixed Concrete Association
Silver Spring, Maryland

Alternate

Jon I. Mullarky, P.E.
National Ready Mixed Concrete Association
Silver Spring, Maryland

Portland Cement Association

Representative

S. K. Ghosh
Portland Cement Association
Skokie, Illinois

Alternate

Joseph J. Messersmith
Portland Cement Association
Rockville, Virginia

Precast/Prestressed Concrete Institute

Representative

Phillip J. Iverson
Precast/Prestressed Concrete Institute
Chicago, Illinois

Alternate

David A. Sheppard
D.A. Sheppard Consulting Structural Engineer, Inc.
Sonora, California

Rack Manufacturers Institute

Representative

Victor Azzi
Rack Manufacturers Institute
Rye, New Hampshire

Alternate

John Nofsinger
Rack Manufacturers Institute
Charlotte, North Carolina

Southern Building Code Congress International

Representative

John Battles
Southern Building Code Congress International
Birmingham, Alabama

Alternate

T. Eric Stafford, E.I.T.
Southern Building Code Congress International
Birmingham, Alabama

Steel Deck Institute

Representative

Bernard E. Cromi
Steel Deck Institute
Fox River Grove, Illinois

Alternate

Richard B. Heagler
Nicholas J. Bouras Inc.
Summit, New Jersey

Structural Engineers Association of Arizona

Representative

Robert Stanley
Structural Engineers Association of Arizona
Scottsdale, Arizona

Structural Engineers Association of California

Representative

Eugene Cole
Cole, Yee, Schubert and Associates
Carmichael, California

Alternate

Thomas Wosser
Degenkolb Engineers
San Francisco, California

Structural Engineers Association of Central California

Representative

Robert N. Chittenden
Division of State Architect
Granite Bay, California

Alternate

Tom H. Hale
Imbsen & Associates
Sacramento, California

Structural Engineers Association of Colorado

Representative

James R. Harris
J.R. Harris and Company
Denver, Colorado

Alternate

Robert B. Hunnes
JVA, Incorporated
Boulder, Colorado

Structural Engineers Association of Illinois

Representative

W. Gene Corley
Construction Technology Laboratories
Skokie, Illinois

Structural Engineers Association of Northern California

Representative

Ronald F. Middlebrook, S.E.
Middlebrook + Louis
San Francisco, California

Alternate

Edwin G. Zacher
H. J. Brunnier Associates
San Francisco, California

Structural Engineers Association of Oregon

Representative

Joseph C. Gehlen
Kramer Gehlen Associates, Inc.
Vancouver, Washington

Alternate

Grant L. Davis
KPFF Consulting Engineers
Portland, Oregon

Structural Engineers Association of Southern California

Representative

Saif Hussain
Saif Hussain & Associates
Woodland Hills, California

Alternate

Saiful Islam
Nabih Youssef and Associates
Los Angeles, California

Structural Engineers Association of San Diego

Representative

Ali Sadre
ESGIL Corporation
San Diego, California

Alternate

Carl Schulze
San Diego, California

Structural Engineers Association of Utah

Representative

Lawrence D. Reaveley
University of Utah
Salt Lake City, Utah

Alternate

Newland Malmquist
Structural Engineers Association of Utah
West Valley City, Utah

Structural Engineers Association of Washington

Representative

James Carpenter
Bruce Olsen Consulting Engineer
Seattle, Washington

Alternate

Bruce C. Olsen
Bruce Olsen Consulting Engineer
Seattle, Washington

The Masonry Society

Representative

John Kariotis
Kariotis and Associates
Sierra Madre, California

Western States Clay Products Association

Representative

Jeff I. Elder, P.E.
Western States Clay Products Association
West Jordan, Utah

Western States Council of Structural Engineers Association

Representative

Greg Shea
Moffatt, Nichol & Bonney, Inc.
Portland, Oregon

Alternate

William T. Rafferty
Structural Design North
Spokane, Washington

Wire Reinforcement Institute, Inc.

Representative

Roy H. Reiterman, P.E.
Wire Reinforcement Institute, Inc.
Findlay, Ohio

Alternate

Robert C. Richardson
Consultant
Sun Lakes, Arizona

FEMA Project Participants

Project Officer

Ugo Morelli
Federal Emergency Management Agency
Washington, D.C.

Project Technical Advisor

Diana Todd
Consulting Engineer
Silver Spring, Maryland

C. Building Seismic Safety Council Project Participants

1996 Board of Direction

Chairman

Eugene Zeller
Director of Planning and Building
Department of Planning and Building
Long Beach, California

Vice Chairman

Richard Wright
Director
Building and Fire Research Laboratory
National Institute of Standards and Technology
Gaithersburg, Maryland
(representing the Interagency Committee for Seismic Safety in Construction)

Secretary

Joseph J. Messersmith
Coordinating Manager
Regional Code Services
Portland Cement Association
Rockville, Virginia

Members

James E. Beavers, Ex officiO
MS Technology, Inc.
Oak Ridge, Tennessee

Mark B. Hogan
Vice President of Engineering
National Concrete Masonry Association
Herndon, Virginia

Nestor Iwankiw
Vice President, Technology and Research
American Institute of Steel Construction
Chicago, Illinois

Gerald H. Jones
Kansas City, Missouri
(representing the National Institute of Building Sciences)

Joseph Nicoletti
Senior Consultant
URS/John A. Blume and Associates
San Francisco, California
(representing the Earthquake Engineering Research Institute)

Clifford J. Ousley
Structural Consultant
Bethlehem Steel Corporation
Bethlehem, Pennsylvania
(representing the American Iron and Steel Institute)

Allan R. Porush
Structural Engineer
Dames and Moore
Los Angeles, California
(representing the Structural Engineers Association of California)

John R. Jack Prosek, Jr., PE
Turner Construction Company
San Francisco, California
(representing the Associated General Contractors of America)

William W. Stewart, FAIA
Stewart Schaberg/Architects
Clayton, Missouri
(representing the American Institute of Architects)

John C. Theiss
Vice President
EQE - Theiss
St. Louis, Missouri
(representing the American Society of Civil Engineers)

Charles H. Thornton
Chairman, The LZA Group, Inc.
Thornton-Tomasetti Engineers
New York, New York
(representing the Applied Technology Council)

David P. Tyree
Regional Manager
American Forest and Paper Association
Georgetown, California

David Wismer
Director of Planning and Code Development
Department of Licenses and Inspections
Philadelphia, Pennsylvania
(representing Building Officials and Code Administrators International)

Staff

James R. Smith, BSSC Executive Director
Thomas R. Hollenbach, BSSC Deputy Executive Director
Claret M. Heider, BSSC Technical Writer-Editor
Mary Marshall, BSSC Administrative Assistant

Rehabilitation Project Oversight Committee

Chairman

Eugene Zeller
Director of Planning and Building
Department of Planning and Building
Long Beach, California

BSSC Members

Gerald H. Jones
Kansas City, Missouri

James R. Smith
BSSC Executive Director
Building Seismic Safety Council
Washington, D.C.

ATC Members

Thomas G. Atkinson
Atkinson, Johnson and Spurrier
San Diego, California

Christopher Rojahn
Applied Technology Council
Redwood City, California

ASCE Members

Paul Seaburg
Office of the Associate Dean
College of Engineering and Technology
Omaha, Nebraska

Ashvin Shah
Director of Engineering
American Society of Civil Engineers
Washington, D.C.

BSSC Project Committee

Chairman

Warner Howe
Consulting Structural Engineer
Germantown, Tennessee

Members

Gerald H. Jones
Kansas City, Missouri

Harry W. Martin
American Iron and Steel Institute
Auburn, California

Allan R. Porush
Structural Engineer
Dames and Moore
Los Angeles, California

F. Robert Preece
Preece/Goudie and Associates
San Francisco, California

William W. Stewart, FAIA
Stewart Schaberg/Architects
Clayton, Missouri

Consultant

Robert A. Olson, President
ROA, Robert Olson Associates, Inc.
Sacramento, California

Seismic Rehabilitation Advisory Panel

Chairman

Gerald H. Jones
Kansas City, Missouri

Members

David E. Allen
Structures Division
Institute of Research in Construction
National Research Council of Canada
Ottawa, Ontario, Canada

John Battles
Southern Building Code Congress, International
Birmingham, Alabama

David C. Breiholz
Chairman, Existing Buildings Committee
Structural Engineers Association of California
Lomita, California

Michael Caldwell
American Institute of Timber Construction
Englewood, Colorado

Gregory Chiu
Insurance Institute for Property Loss Reduction
Boston, Massachusetts

Terry Dooley
Morley Construction Company
Santa Monica, California

Susan M. Dowty
International Conference of Building Officials
Whittier, California

Steven J. Eder
EQE Engineering Consultants
San Francisco, California

S. K. Ghosh
Portland Cement Association
Skokie, Illinois

Barry J. Goodno
Professor
School of Civil Engineering
Georgia Institute of Technology
Atlanta, Georgia

Charles C. Gutberlet
U.S. Army Corps of Engineers
Washington, D.C.

Warner Howe
Consulting Structural Engineer
Germantown, Tennessee

Howard Kunreuther
Wharton School
University of Pennsylvania
Philadelphia, Pennsylvania

Harry W. Martin
American Iron and Steel Institute
Auburn, California

Robert McCluer
Building Officials and Code Administrators, International
Country Club Hills, Illinois

Margaret Pepin-Donat
National Park Service Retired
Edmonds, Washington

William Petak
Professor, Institute of Safety and Systems Management
University of Southern California
Los Angeles, California

Howard Simpson
Simpson, Gumpertz and Heger
Arlington, Massachusetts

William W. Stewart, FAIA
Stewart Schaberg/Architects
Clayton, Missouri

James E. Thomas
Duke Power Company
Charlotte, North Carolina

L. Thomas Tobin
Tobin and Associates
Mill Valley, California

Representatives of BSSC Member Organizations and their Alternates
(as of September 1996)

AFL-CIO Building and Construction Trades Department

Representative
Bradley Sant
AFL-CIO/Building and Construction Trades Department
Washington, D.C.

Alternate
Pete Stafford
Center to Protect Workers' Rights
Washington, D.C.

AISC Marketing, Inc.

Representative
Robert Pyle
AISC Marketing, Inc.
Buena Park, California

American Concrete Institute

Representative
Arthur J. Mullkoff
Staff Engineer
American Concrete Institute
Detroit, Michigan

Alternate
Ward R. Malisch
Managing Director, Engineering
American Concrete Institute
Detroit, Michigan

American Consulting Engineers Council

Representative
Roy G. Johnston
Structural Engineer
Brandow and Johnston Associates
Los Angeles, California

Alternate
Edward Bajer
Director of Energy and Interprofessional Development
American Consulting Engineers Council
Washington, D.C.

American Forest and Paper Association

Representative
David P. Tyree, P.E.
Regional Manager
American Forest and Paper Association
Georgetown, California

Alternate
Bradford K. Douglas, P.E.
Director of Engineering
American Forest and Paper Association
Washington, D.C.

American Institute of Architects

Representative
William W. Stewart
William W. Stewart, FAIA
Stewart Schaberg/Architects
Clayton, Missouri

Alternate
Gabor Lorant
Gabor Lorant Architect, Inc.
Phoenix, Arizona

American Institute of Steel Construction

Representative
Nestor Iwankiw
Vice President, Technology and Research
American Institute of Steel Construction
Chicago, Illinois

American Insurance Services Group, Inc.

Representative
John A. Mineo
Manager, Construction
American Insurance Services Group, Inc.
New York, New York

Alternate
Phillip Olmstead
Senior Technical Consultant
ITT Hartford Insurance Group
Hartford, Connecticut

American Iron and Steel Institute

Representative
Harry W. Martin
Regional Director
American Iron and Steel Institute
Newcastle, California

Alternate
Clifford J. Ousley
Bethlehem Steel Corporation
Bethlehem, Pennsylvania

American Plywood Association

Representative
Kenneth R. Andreason
American Plywood Association
Tacoma, Washington

Alternate
William A. Baker
Manager, Market Support Services
American Plywood Association
Tacoma, Washington

American Society of Civil Engineers

Representative
John C. Theiss
Vice President
EQE - Theiss
St. Louis, Missouri

Alternate
Ashvin Shah
American Society of Civil Engineers
Washington, D.C.

American Society of Civil Engineers - Kansas City Chapter

Representative
Harold Sprague
Kansas City Chapter of ASCE
Kansas City, Missouri

Alternate
Brad Vaughan
Kansas City Chapter of ASCE
Kansas City, Missouri

American Society of Heating, Refrigerating and Air-Conditioning Engineers, Inc.

Representative
William Staehlin
Chairman
ASHRAE Task Group on Seismic Restraint Design
Sacramento, California

Alternate
J. Richard Wright
Director of Technology
ASHRE
Atlanta, Georgia

American Society of Mechanical Engineers

Representative
June Yarmush
Engineering Advisor
ASME
New York, New York

American Welding Society

Representative
Hardy C. Campbell III
Senior Engineer
American Welding Society
Miami, Florida

Alternate
Charles R. Fassinger
Managing Director, Technical Services
American Welding Society
Miami, Florida

Applied Technology Council

Representative
Christopher Rojahn
Executive Director
Applied Technology Council
Redwood City, California

Alternate
Charles N. Thornton
Chairman, The LZA Group, Inc.
Thornton-Tomasetti
New York, New York

Associated General Contractors of America

Representative
John R. Jack Prosek, Jr., PE
Turner Construction Company
San Francisco, California

Alternate
Christopher Monek
Associated General Contractors of America
Washington, D.C.

Association of Engineering Geologists

Representative
Ellis Krinitzsky
Army Corps of Engineers
Waterways Experiment Station
Vicksburg, Mississippi

Alternate
Patrick J. Barosh
Patrick J. Barosh and Associates
Concord, Massachusetts

Association of Major City Building Officials

Representative
Arthur J. Johnson, Jr.
City of Los Angeles
Department of Building and Safety
Los Angeles, California

Alternate
Karl Deppe
City of Los Angeles
Department of Building and Safety
Los Angeles, California

Brick Institute of America

Representative
J. Gregg Borchelt
Director of Engineering and Research
Brick Institute of America
Reston, Virginia

Alternate
Matthew Scolforo
Brick Institute of America
Reston, Virginia

Building Officials and Code Administrators International

Representative
David Wismer
Director of Planning and Code Development
Department of Licenses and Inspections
Philadelphia, Pennsylvania

Alternate
Paul K. Heilstedt
Chief Executive Officer
BOCA, International
Country Club Hills, Illinois

Building Owners and Managers Association International

Representative
Michael Jawer
BOMA, International
Washington, D.C.

California Geotechnical Engineers Association

Representative
Alan Kropp
Alan Kropp and Associates
Berkeley, California

Alternate
John A. Baker
Anderson Geotechnical Consultants
Roseville, California

Canadian National Committee on Earthquake Engineering

Representative
R. H. Devall
Chair, Canadian National Committee on Earthquake Engineering
Vancouver, B.C.

Alternate
D. A. Lutes
Canadian National Committee on Earthquake Engineering
National Research Council of Canada
Division of Research Building
Ottawa, Ontario, Canada

Concrete Masonry Association of California and Nevada

Representative
Stuart R. Beavers
Executive Director
Concrete Masonry Association of California and Nevada
Citrus Heights, California

Alternate
Daniel Shapiro
Principal
SOH and Associates, Structural Engineers
San Francisco, California

Concrete Reinforcing Steel Institute

Representative
David P. Gustafson
Concrete Reinforcing Steel Institute
Schaumburg, Illinois

Alternate
H. James Nevin
Western Regional Director
Concrete Reinforcing Steel Institute
Glendora, California

Earthquake Engineering Research Institute

Representative
Joseph Nicoletti
URS Consultants
San Francisco, California

Alternate
F. Robert Preece
Preece/Goudie and Associates
San Francisco, California

Insurance Institute for Property Loss Reduction

Representative
Gregory L. F. Chiu
Engineer
Insurance Institute for Property Loss Reduction
Boston, Massachusetts

Alternate
Karen Gahagan
Assistant Vice President for Information Services
Insurance Institute for Property Loss Reduction
Boston, Massachusetts

Interagency Committee on Seismic Safety in Construction

Representative
Richard Wright
Director
Building and Fire Research Laboratory
National Institute of Standards and Technology
Gaithersburg, Maryland

Alternate
H. S. Lew
National Institute of Standards and Technology
Building and Fire Research Laboratory
Gaithersburg, Maryland

International Conference of Building Officials

Representative
Rick Okawa
International Conference of Building Officials
Whittier, California

Alternate
Susan M. Dowty
Senior Staff Engineer
International Conference of Building Officials
Whittier, California

Masonry Institute of America

Representative
John Chrysler
Executive Director
Masonry Institute of America
Los Angeles, California

Alternate
James E. Amrhein
Masonry Institute of America
Los Angeles, California

Metal Building Manufacturers Association

Representative
W. Lee Shoemaker, Ph.D., P. E.
Metal Building Manufacturers Association
Cleveland, Ohio

Alternate
Joe N. Nunnery
AMCA Buildings Division
Memphis, Tennessee

National Association of Home Builders

Representative
Robert Elliott
Construction Technology and Codes Specialist
National Association of Home Builders
Washington, D.C.

National Concrete Masonry Association

Representative
Mark B. Hogan, P.E.
Vice President of Engineering
National Concrete Masonry Association
Herndon, Virginia

Alternate
Phillip J. Samblanet
Structural Engineer
National Concrete Masonry Association
Herndon, Virginia

National Conference of States on Building Codes and Standards

Representative
Richard T. Conrad, AIA
California Building Standards Commission
Sacramento, California

Alternate
Robert C. Wible
Executive Director
National Conference of States on Building Codes and Standards
Herndon, Virginia

National Council of Structural Engineers Associations

Representative
Howard Simpson
Simpson, Gumpertz and Heger
Arlington, Massachusetts

Alternate
W. Gene Corley
Vice President
Construction Technology Laboratories
Skokie, Illinois

National Elevator Industry, Inc.

Representative
George A. Kappenhagen
Schindler Elevator Corporation
Morristown, New Jersey

National Fire Sprinkler Association, Inc.

Representative
Russell P. Fleming
Director, Engineering and Standards
National Fire Sprinkler Association
Patterson, New York

Alternate
Kenneth E. Isman
Associate Director of Engineering and Standards
National Fire Sprinkler Association
Patterson, New York

National Institute of Building Sciences

Representative
Gerald H. Jones
Kansas City, Missouri

National Ready Mixed Concrete Association

Representative
Robert A. Garbini
National Ready Mixed Concrete Association
Silver Spring, Maryland

Alternate
Jon I. Mullarky
First Vice President
National Ready Mixed Concrete Association
Silver Spring, Maryland

Portland Cement Association

Representative
S. K. Ghosh
Director, Engineering Services, Codes and Standards
Portland Cement Association
Skokie, Illinois

Alternate
Joseph J. Messersmith
Portland Cement Association
Rockville, Virginia

Precast/Prestressed Concrete Institute

Representative
Phillip J. Iverson
Technical Director
Precast/Prestressed Concrete Institute
Chicago, Illinois

Alternate
David A. Sheppard
D.A. Sheppard Consulting Structural Engineer, Inc.
Sonora, California

Rack Manufacturers Institute

Representative
Victor Azzi
Rack Manufacturers Institute
Rye, New Hampshire

Alternate
John Nofsinger
Managing Director
Rack Manufacturers Institute
Charlotte, North Carolina

California Seismic Safety Commission

Representative
Fred Turner
Staff Structural Engineer
Seismic Safety Commission (California)
Sacramento, California

Southern Building Code Congress International

Representative
John Battles
Manager/Codes
Southern Building Code Congress, International
Birmingham, Alabama

Steel Deck Institute, Inc.

Representative
Bernard E. Cromi
Managing Director
Steel Deck Institute, Inc.
Canton, Ohio

Structural Engineers Association of Arizona

Representative
Tom Griffis
Structural Engineers Association of Arizona
Scottsdale, Arizona

Structural Engineers Association of California

Representative
Allan R. Porush
Dames and Moore
Los Angeles, California

Alternate
Thomas Wosser
Degenkolb Engineers
San Francisco, California

Structural Engineers Association of Central California

Representative
Robert N. Chittenden
Structural Engineers Association of Central California
Fair Oaks, California

Alternate
Tom H. Hale
Cole, Yee, Schubert and Associates
Sacramento, California

Structural Engineers Association of Colorado

Representative
James R. Harris
President
J. R. Harris and Company
Denver, Colorado

Alternate
Robert B. Hunnes
President
JVA, Inc.
Boulder, Colorado

Structural Engineers Association of Illinois

Representative
W. Gene Corley
Vice President
Construction Technology Laboratories
Skokie, Illinois

Structural Engineers Association of Northern California

Representative
Ronald Hamburger
President
Structural Engineers Association of Northern California
San Francisco, California

Alternate
Edwin G. Zacher
H. J. Brunnier Associates
San Francisco, California

Structural Engineers Association of Oregon

Representative
Joseph C. Gehlen
Kramer Gehlen Associates, Inc.
Vancouver, Washington

Alternate
Grant L. Davis
Structural Engineers Association of Oregon
Portland, Oregon

Structural Engineers Association of Southern California

Representative
Saif Hussain
Principal, Saif Hussain and Associates
Woodland Hills, California

Structural Engineers Association of San Diego

Representative
Ali Sadre
Structural Engineers Association of San Diego
San Diego, California

Structural Engineers Association of Utah

Representative
Lawrence D. Reaveley
Civil Engineering Department
University of Utah
Salt Lake City, Utah

Alternate
Newland Malmquist
Structural Engineers Association of Utah
West Valley City, Utah

Structural Engineers Association of Washington

Representative
James Carpenter
Bruce Olsen Consulting Engineer
Seattle, Washington

Alternate
Bruce C. Olsen
Bruce Olsen Consulting Engineer
Seattle, Washington

The Masonry Society

Representative
John Kariotis
President
Kariotis and Associates, Structural Engineers, Inc.
South Pasadena, California

Western States Clay Products Association

Representative
Donald A. Wakefield, P.E.
Western States Clay Products Association
Sandy, Utah

Western States Council of Structural Engineers Association

Representative
Roger McGarrigle
Western States Council of Structural Engineers Association
Portland, Oregon

Alternate
William T. Rafferty
Western States Council of Structural Engineers Association
Spokane, Washington

US CUSTOMARY TO SI UNIT CONVERSION TABLES

	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

Index