NIST GCR 10-917-7

Program Plan for the Development of Collapse Assessment and Mitigation Strategies for Existing Reinforced Concrete Buildings

NEHRP Consultants Joint Venture

A partnership of the Applied Technology Council and the Consortium of Universities for Research in Earthquake Engineering

nehrp

National Institute of Standards and Technology
U.S. Department of Commerce

Disclaimers

The policy of the National Institute of Standards and Technology is to use the International System of Units (metric units) in all of its publications. However, in North America in the construction and building materials industry, certain SI units are not widely used such that it is more practical and less confusing to include measurement values for customary units only.

Cover photo – 1999 Koceali (Turkey) earthquake (courtesy of NISEE Earthquake Engineering Online Archive)

NIST GCR 10-917-7

Program Plan for the Development of Collapse Assessment and Mitigation Strategies for Existing Reinforced Concrete Buildings

Prepared for
U.S. Department of Commerce
Building and Fire Research Laboratory
National Institute of Standards and Technology
Gaithersburg, MD 20899-8600

By
NEHRP Consultants Joint Venture
A partnership of the Applied Technology Council and the
Consortium of Universities for Research in Earthquake Engineering

August 2010

U.S. Department of Commerce
Gary Locke, Secretary

National Institute of Standards and Technology
Patrick D. Gallagher, Director

NIST GCR 10-917-7

Participants

National Institute of Standards and Technology

John (Jack) R. Hayes, Director - National Earthquake Hazards Reduction Program
Jeff Dragovich, Project Manager

NEHRP Consultants Joint Venture

Applied Technology Council
201 Redwood Shores Parkway, Suite 240
Redwood City, California 94065
www.ATCouncil.org

Consortium of Universities for
Research in Earthquake Engineering
1301 S. 46th Street - Bldg. 420
Richmond, California 94804
www.CUREE.org

Joint Venture Management Committee

James R. Harris
Robert Reitherman
Christopher Rojahn
Andrew Whittaker

Joint Venture Program Committee

Jon A. Heintz (Program Manager)
Michael Constantinou
C.B. Crouse
James R. Harris
William T. Holmes
Jack Moehle
Andrew Whittaker

Project Technical Committee

Ken Elwood (Project Director)
Craig Comartin
William T. Holmes
Dominic Kelly
Laura Lowes
Jack Moehle

Project Review Panel

Nathan Gould
Afshar Jalalian
Jim Jirsa
Terry Lundeen
Mike Mehrain
Julio Ramirez

Project Manager

David Hutchinson

Preface

The NEHRP Consultants Joint Venture is a partnership between the Applied Technology Council (ATC) and the Consortium of Universities for Research in Earthquake Engineering (CUREE). In 2007, the National Institute of Standards and Technology (NIST) awarded a National Earthquake Hazards Reduction Program (NEHRP) "Earthquake Structural and Engineering Research" contract (SB1341-07-CQ-0019) to the NEHRP Consultants Joint Venture to conduct a variety of tasks, including Task Order 69297 entitled "Integration of Collapse Risk Mitigation Standards and Guidelines for Older Reinforced Concrete Buildings into National Standards: Phase I." The objective of this project was to develop a program plan for establishing nationally accepted guidelines for assessing and mitigating risks in older concrete buildings.

Work on this project was intended to be an extension of a National Science Foundation (NSF), George E. Brown, Jr. Network for Earthquake Engineering Simulation (NEES) Grand Challenge project, "Mitigation of Collapse Risks in Older Reinforced Concrete Buildings," being conducted by the Pacific Earthquake Engineering Research (PEER) Center. The purpose of the Grand Challenge project is to utilize NEES resources in developing comprehensive strategies for identifying seismically hazardous older concrete buildings and promoting effective hazard mitigation strategies for those buildings found to be at risk of collapse. Results from the NEES Grand Challenge project are expected to be directly applicable to the long-term objectives of this project.

This report is intended to provide the basis of a multi-phase program for the development of nationally accepted guidelines for the collapse prevention of older nonductile concrete buildings. It summarizes the scope and content of a series recommended guidance documents, the necessary analytical studies, and estimated costs associated with their development.

The NEHRP Consultants Joint Venture is indebted to the leadership of Dave Hutchinson, Project Manager, Ken Elwood, Project Director, and to the members of the Project Technical Committee, consisting of Craig Comartin, Bill Holmes, Dominic Kelly, Laura Lowes and Jack Moehle for their contributions in developing this report and the resulting recommendations. The Project Review Panel, consisting of Nathan Gould, Afshar Jalalian, Jim Jirsa, Terry Lundeen, Mike Mehrain and Julio Ramirez, provided technical review and commentary at key developmental milestones on the project. The names and affiliations of all who contributed to this report are provided in the list of Project Participants.

The NEHRP Consultants Joint Venture also gratefully acknowledges Jack Hayes and Jeff Dragovich (NIST) for their input and guidance in the preparation of the report, and Peter Mork (ATC) for report production services.

Jon A. Heintz
Program Manager

Table of Contents

List of Figures

List of Tables

Executive Summary

Reinforced concrete buildings designed and constructed prior to the introduction of seismic design provisions for ductile response (commonly referred to as *nonductile* concrete buildings) represent one of the largest seismic safety concerns in the United States and the world. The need for improvement in collapse assessment technology for existing nonductile concrete buildings has been recognized as a high-priority because: (1) such buildings represent a significant percentage of the vulnerable building stock across the United States; (2) failure of such buildings can involve total collapse, substantial loss of life, and significant economic loss; (3) at present, the ability to predict collapse thresholds for different types of older reinforced concrete buildings is limited; (4) recent research has focused on older West Coast concrete buildings; and, (5) full advantage has not yet been taken of past research products (ATC, 2003).

The National Science Foundation awarded a George E. Brown, Jr. Network for Earthquake Engineering Simulation (NEES) Grand Challenge project to the Pacific Earthquake Engineering Research (PEER) Center to develop comprehensive strategies for identifying seismically hazardous older concrete buildings, enable prediction of the collapse of such buildings, and to develop and promote cost-effective hazard mitigation strategies for them. Products from this important research effort are expected to soon be available, creating an opportunity for transferring past and present research results into design practice.

Recognizing this opportunity, the National Institute of Standards and Technology (NIST) has initiated a multi-phase project with the primary objective being the development of nationally accepted guidelines for assessing and mitigating the risk of collapse in older nonductile concrete buildings. This report summarizes efforts to define the scope and content of recommended guidance documents, the necessary analytical studies, and estimated schedule and budget needed for their development.

Based on limitations in current seismic evaluation and rehabilitation practice in the United States (Chapter 2), a review of information currently being developed in the NEES Grand Challenge project (Chapter 3), and an understanding of common deficiencies found in nonductile concrete buildings (Chapter 4), the following critical needs for addressing the collapse risk associated with older concrete construction have been identified:

- Improved procedures for identifying building systems vulnerable to collapse, including simple tools that do not require detailed analysis.

- Updated acceptance criteria for concrete components based on latest research results.
- Identification of cost-effective mitigation strategies to reduce collapse risk in existing concrete buildings.

To address these needs, the development of a series of guidance documents is recommended (Chapter 5). Under the umbrella title *Guidance for Collapse Assessment and Mitigation Strategies for Existing Reinforced Concrete Buildings*, the first document is intended to focus on building system behavior, while the remaining documents focus on individual concrete components. As currently envisioned, the series comprises the following eight documents; however, other documents could be conceived in the future to extend the series and address future developing needs:

1. Assessment of Collapse Potential and Mitigation Strategies
2. Acceptance Criteria and Modeling Parameters for Concrete Components: Columns
3. Acceptance Criteria and Modeling Parameters for Concrete Components: Beam-Column Joints
4. Acceptance Criteria and Modeling Parameters for Concrete Components: Slab-Column Systems
5. Acceptance Criteria and Modeling Parameters for Concrete Components: Walls
6. Acceptance Criteria and Modeling Parameters for Concrete Components: Infill Frames
7. Acceptance Criteria and Modeling Parameters for Concrete Components: Beams
8. Acceptance Criteria and Modeling Parameters for Concrete Components: Rehabilitated Components

A potential methodology for identifying parameters correlated with an elevated probability of collapse based on results of comprehensive collapse simulations and estimation of collapse probabilities for a collection of building prototypes is described (Chapter 6). For consistency between all documents, a common developmental methodology is recommended for the selection of acceptance criteria and modeling parameters (Chapter 7).

The risk associated with older nonductile concrete buildings in the United States is significant, and the development of improved technologies for mitigating that risk is a large undertaking. A multi-phase, multi-year effort is needed to complete all eight recommended guidance documents (Chapter 8). A modular approach to the work plan has been structured to provide flexibility in funding and scheduling the various components of the recommended program.

With the assumption that no more than two component documents are under development at any one time, the overall program has a duration of seven years. In general, work can be conducted in parallel or in series, as funding permits. Some coordination between phases, however, is recommended. The development of Document 1 is considered the greatest need, and is recommended as the highest priority. It has been structured to be completed in phases, with an overall duration of five years.

The estimated budget for the overall program is $5.2 million. The estimated budget for the development of Document 1 is $2.9 million, which is the total for Phase 1 ($900,000), Phase 2 ($700,000), and Phase 3 ($1,300,000).

The problem associated with older nonductile concrete buildings has attracted the attention of a number of stakeholders who are potential collaborators on the implementation of this work plan. Successful development of the recommended guidance documents should include collaboration with these stakeholders, some of which will be providers of necessary information, or sources of supplemental funding.

Chapter 1

Introduction

Reinforced concrete buildings designed and constructed prior to the introduction of seismic design provisions for ductile response (commonly referred to as *nonductile* concrete buildings) represent one of the largest seismic safety concerns in the United States and the world. The California Seismic Safety Commission (1995) states, "many older concrete frame buildings are vulnerable to sudden collapse and pose serious threats to life." The poor seismic performance of nonductile concrete buildings is evident in recent earthquakes, including: Northridge, California (1994); Kobe, Japan (1995); Chi Chi, Taiwan (1999); Izmit, Düzce, and Bingol Turkey (1999, 1999, 2003); Sumatra (2004); Pakistan (2005); China (2008); Haiti (2010); and Chile (2010).

The exposure to life and property loss in a major earthquake near an urban area is immense. Nonductile concrete buildings include residential, commercial, critical business, and essential (emergency) services, and many are high occupancy structures. Partial or complete collapse of nonductile concrete structures can result in significant loss of life. Severe damage can lead to loss of critical building contents and functionality, and risk of financial ruin for business occupancies. Without proactive steps to understand and address these types of structures, the risks they pose will persist.

The *Concrete Coalition*, a joint project of the Earthquake Engineering Research Institute, the Applied Technology Council and the Pacific Earthquake Engineering Research (PEER) Center, is a network of individuals, governments, institutions, and agencies with an interest in assessing the risk associated with nonductile concrete buildings and promoting the development of policies and procedures for mitigating that risk. Initially, the effort has focused on estimating the number of nonductile concrete buildings in highly seismic areas of California.

On the basis of detailed surveys and extrapolation across California, the Concrete Coalition (2010) estimates there are approximately 1,500 pre-1980 concrete buildings in the City of Los Angeles, 3,000 in San Francisco, and 20,000 in the 33 most seismically active counties state-wide. Outside of California, nonductile concrete buildings are widespread nationally and worldwide. These numbers portend the scale of the problem nationally and globally, where nonductile concrete buildings are more prevalent.

Based on these initial efforts and interactions with various stakeholders, the Concrete Coalition has identified an emerging critical need to begin development of more efficient procedures for assessing the collapse potential of nonductile concrete buildings and identifying particularly dangerous buildings for detailed evaluation and retrofit.

Evidence from earthquake reconnaissance efforts world-wide shows that strong earthquakes can result in a wide range of damage to nonductile concrete buildings, ranging from minor cracking to collapse (Otani 1999). Current guidelines and standards for seismic assessment of existing concrete buildings are not sufficiently refined to enable engineers to quickly and reliably distinguish between buildings that might be expected to collapse and those that might sustain moderate to severe damage. As a consequence, engineers have tended toward conservative practices, and guidelines and standards for seismic evaluation and rehabilitation have tended to be conservative.

Conservative evaluation techniques applied to nonductile concrete buildings almost always indicate that there is a risk of collapse, and that extensive rehabilitation is needed to mitigate that risk. Recent policy efforts demonstrate the difficulties in legislating large-scale retrofit programs encompassing nonductile concrete buildings without adequate resources or reliable engineering tools. In the case of the California hospital retrofit program (OSHPD 2009), almost all nonductile concrete buildings were categorized as high risk, needing costly retrofit.

Considering the challenges and limitations associated with funding seismic rehabilitation, this situation (thousands of buildings, nearly all classified as high risk) is not tenable. This “always bad” message is not credible, and fosters an environment in which retrofitting of concrete buildings at risk of collapse is not happening quickly enough. To achieve a meaningful reduction in the seismic risk posed by nonductile concrete buildings, there is a need for guidelines that can reliably identify the subset of buildings that are most vulnerable to collapse, and that provide cost-effective retrofit solutions for these buildings.

In 2006, the National Science Foundation (NSF) awarded a George E. Brown, Jr. Network for Earthquake Engineering Simulation (NEES) Grand Challenge project, *Mitigation of Collapse Risks in Older Reinforced Concrete Buildings*, to the Pacific Earthquake Engineering Research (PEER) Center. The Grand Challenge project is tasked with using NEES resources to develop comprehensive strategies for identifying seismically hazardous older concrete buildings, enabling prediction of the collapse of such buildings, and developing and promoting cost-effective hazard mitigation strategies for them. While the *Grand Challenge* research project is expected to develop new knowledge about these buildings, it is anticipated that additional applied research and technology transfer activities will be needed to transition this knowledge into guidelines that can be used in engineering practice.

Recognizing this opportunity, National Institute of Standards and Technology (NIST) initiated a project with the primary objective being the development of nationally accepted guidelines for the assessment and mitigation of collapse risk in older reinforced concrete buildings. This report summarizes efforts to define the scope and content of recommended guidance documents, the necessary analytical studies, and estimated schedule and budget needed for their development. The report is organized as follows:

- Chapter 2 summarizes the current state-of-practice with regard to seismic evaluation and rehabilitation, and identifies limitations in currently available assessment procedures.
- Chapter 3 summarizes research being conducted on the NEES Grand Challenge project, and describes experimental testing and analytical studies that are relevant to future recommended work.
- Chapter 4 summarizes common deficiencies found in nonductile concrete buildings and retrofit strategies typically used to address these vulnerabilities.
- Chapter 5 provides an overview of a series of recommended guidance documents to be developed under the umbrella title *Guidance for Collapse Assessment and Mitigation Strategies for Existing Reinforced Concrete Buildings*.
- Chapter 6 outlines focused analytical studies needed to establish limits on parameters that influence the collapse vulnerability of nonductile concrete buildings.
- Chapter 7 describes a methodology for developing improved acceptance criteria and modeling parameters for concrete components.
- Chapter 8 summarizes recommended work plan tasks, schedule, and estimated costs for a multi-year program to develop the recommended guidance documents, and lists key collaborators that should be involved in such a program.

This report and the recommendations herein focus on cast-in-place concrete construction. While existing precast concrete buildings also pose a risk of collapse in earthquakes, collapse behavior of precast concrete construction is significantly different from cast-in-place concrete buildings. Given the substantial technical differences associated with segmented construction and precast connection vulnerability, treatment of precast concrete buildings has been excluded from consideration in this program. This exclusion is not meant to imply that additional study of the collapse vulnerability of existing precast concrete buildings is unimportant, or should not be undertaken. It is recommended that future funding be focused on addressing the risk of precast concrete buildings separately and specifically.

Chapter 2

Summary and Limitations of Current Seismic Evaluation and Rehabilitation Practice

This chapter lists currently available resources for seismic evaluation and rehabilitation, describes regional variations in U.S. engineering practice, and identifies limitations in key resources related to the identification of collapse-vulnerable nonductile concrete buildings.

2.1 Selected Resources

In the United States, there are many different approaches used to assess the seismic resistance of buildings. Currently available engineering resources take the form of guidelines, standards, national model building codes, and institutional policies. Selected resources include the following:

- FEMA 154, *Rapid Visual Screening of Buildings for Potential Seismic Hazards: A Handbook*, Second Edition (FEMA, 2002)
- American Society of Civil Engineers, ASCE/SEI 7, *Minimum Design Loads for Buildings and Other Structures* (ASCE, 2006)
- American Society of Civil Engineers, ASCE/SEI 31, *Seismic Evaluation of Existing Buildings* (ASCE, 2003)
- American Society of Civil Engineers, ASCE/SEI 41, *Seismic Rehabilitation of Existing Buildings*, (ASCE, 2007a)
- International Code Council (ICC), *International Building Code* (ICC, 2009a)
- International Code Council (ICC), *International Existing Building Code* (ICC, 2009b)
- National Institute of Standards and Technology, ICSSC RP 6/NISTIR 6762, *Standards of Seismic Safety for Existing Federally Owned and Leased Buildings*, ICSSC RP 6/NISTIR 6762 (NIST, 2002)
- Department of Defense, Unified Facilities Criteria (UFC) 3-330-03A, *Seismic Review Procedures for Existing Military Buildings* (DOD, 2005)
- Department of Defense, Unified Facilities Criteria (UFC) 3-300-10N, *Structural Engineering* (DOD, 2006)

- Department of Defense, Unified Facilities Criteria (UFC) 3-310-04, *Seismic Design for Buildings* (DOD, 2007)

Much of the practice for seismic evaluation and rehabilitation in the United States is based on ASCE/SEI 31 *Seismic Evaluation of Existing Buildings* and ASCE/SEI 41 *Seismic Rehabilitation of Existing Buildings*. In some cases, evaluation and rehabilitation is based on a percentage of the strength required in codes and standards for new buildings, such as the *International Building Code* and ASCE/SEI 7 *Minimum Design Loads for Buildings and Other Structures*. Federal, state, and private institutional policies often refer to some combination of the above resources.

Worldwide, several additional assessment and rehabilitation standards and guidelines are used, including *Eurocode 8: Design of Structures for Earthquake Resistance – Part 3: Assessment and Retrofitting of Buildings* (CEN, 2005) in Europe, *Assessment and Improvement of the Structural Performance of Buildings in Earthquakes* (NZSEE, 2006) in New Zealand, and *Standard for Evaluation of Existing Reinforced Concrete Buildings* (JBDPA, 2001) in Japan. Many international approaches are similar in concept to ASCE/SEI 41.

2.2 Initiation of Seismic Evaluation and Rehabilitation Work

Seismic evaluation and rehabilitation work on existing buildings is initiated in one of three ways. Efforts are mandated, triggered, or voluntarily undertaken (ATC, 2009b):

- Mandated programs are those that require seismic rehabilitation (or at least evaluation) for specified buildings regardless of action on the part of a building owner.
- In triggered programs, seismic evaluation or rehabilitation might not be intended on the part of the building owner, but is required (or triggered) based on the scope of repairs, additions, alterations, changes in occupancy, or other work that is being performed on a building.
- Voluntary rehabilitation is work initiated by the building owner (or other stakeholder) and subject to minimal outside requirements. Voluntary work is generally driven by institutional policy or the risk sensitivity of an individual building owner. Although full compliance is not required or necessary, codes and standards are often used to guide seismic evaluation and design as part of voluntary rehabilitation efforts.

Commercial, institutional, state, and local government buildings are regulated by a local authority having jurisdiction in an area. Most local building codes are based on a national model building code such as the *International Building Code* (IBC). Triggers for seismic work on existing buildings that are undergoing repairs, alterations, additions, or changes in use are contained in Chapter 34 of the IBC, or in the *International Existing Building Code* (IEBC), where the IEBC has been adopted.

In Chapter 34 of the IBC, equivalent lateral force provisions for new buildings are applied to existing buildings, but with some relaxation of component detailing requirements. The IEBC contains provisions that are similar, but also permits the use of ASCE/SEI 31 and ASCE/SEI 41 for evaluation and rehabilitation.

The General Services Administration (GSA) requires seismic evaluation of federal buildings that are being considered for purchase, lease, renovation, or expansion. The GSA specifies the use of ICSSC RP 6/NISTIR 6762 for minimum seismic requirements. ICSSC RP 6/NISTIR 6762 refers to ASCE/SEI 31 and ASCE/SEI 41 for evaluation and rehabilitation criteria.

The Department of Defense requires the use of the Unified Facilities Criteria (UFC), which is a series of documents that provide planning, design, construction, sustainment, restoration, and modernization criteria for building structures. UFC 3-300-10N *Structural Engineering* refers to ICSSC RP 6/NISTIR 6762. UFC 3-310-04 *Seismic Design for Buildings* also refers to ICSSC RP 6/NISTIR 6762, but also directly requires the use of ASCE/SEI 31 and ASCE/SEI 41 for seismic evaluation and rehabilitation of existing buildings.

2.3 Regional Variations in Engineering Practice

There are significant regional variations in the seismic evaluation and rehabilitation of existing buildings based on differences in the political, jurisdictional, economic, and seismic realities across the United States (ATC, 2009b). Areas that are subjected to relatively frequent earthquakes, such as the Western United States, possess a much greater awareness of seismic risk than areas that have not experienced a significant, damaging earthquake in recent memory, such as the Central and Eastern United States. This awareness affects how seismic evaluation and rehabilitation projects are initiated in different regions.

2.3.1 Western U.S. Practice

In the Western United States, especially in regions of high seismicity, seismic considerations are an integral part of structural design practice, and engineers are frequently engaged in seismic projects (ATC, 2009b). There are numerous examples of mandated seismic programs targeting a specific type of construction (e.g., unreinforced masonry buildings) or occupancy (e.g., essential hospital facilities). State and local building codes include triggers for seismic work that are related to repairs, additions, alterations, or changes in occupancy, and such triggers are routinely enforced. Many individual building owners, corporations, and institutions have initiated voluntary programs to minimize their exposure to seismic risk, and seismic evaluation and rehabilitation projects are regularly performed.

2.3.2 Central and Eastern U.S. Practice

In the Central and Eastern United States, especially in regions of moderate and low seismicity, seismic evaluation and rehabilitation work is rarely performed. Mandated seismic programs are almost nonexistent. Where seismic rehabilitation does occur, it is largely triggered by additions, alterations, or changes in use or occupancy, and is met with significant resistance (ATC, 2009b). Notable exceptions to this include voluntary seismic work that is initiated by federal agencies or large national or multi-national private corporations as part of building acquisition, maintenance, and renovation activities.

Large private corporations often have a presence in regions of high seismicity, and are familiar with the seismic risks associated with older buildings in their portfolio. Often such corporations will evaluate buildings in regions of moderate seismicity, but will exempt buildings in regions of low seismicity. Seismic rehabilitation of commercial and institutional buildings in regions of moderate and low seismicity is often not triggered by applicable building codes. If triggered, the requirements are often not enforced.

In the case of federal buildings, ICSSC RP 6/NISTIR 6762 requires existing buildings in regions of moderate and low seismicity to be treated similar to buildings in regions of high seismicity. Federal buildings that are located in regions of very low seismicity are exempted.

2.4 Reference Standards for Seismic Evaluation and Rehabilitation of Existing Buildings

Prevailing practice for seismic evaluation and rehabilitation in the United States is based on ASCE/SEI 31 *Seismic Evaluation of Existing Buildings* and ASCE/SEI 41 *Seismic Rehabilitation of Existing Buildings*. These standards are the most commonly used, especially in regions of high seismicity. They have been specified in mandatory seismic mitigation programs, are currently referenced in model building codes when seismic work is triggered, and are frequently cited as criteria in voluntary retrofit projects or institutional programs.

2.4.1 ASCE/SEI 31 Standard for Seismic Evaluation of Existing Buildings

ASCE/SEI 31 is a national consensus standard applicable to the evaluation of structural and nonstructural performance levels of Life Safety and Immediate Occupancy at any level of seismicity. As illustrated in Figure 2-1, the methodology contained within ASCE/SEI 31 was initially developed in the mid-1980s, and is based on a series of predecessor documents dating back to ATC-14, *Evaluating the Seismic Resistance of Existing Buildings* (ATC, 1987).

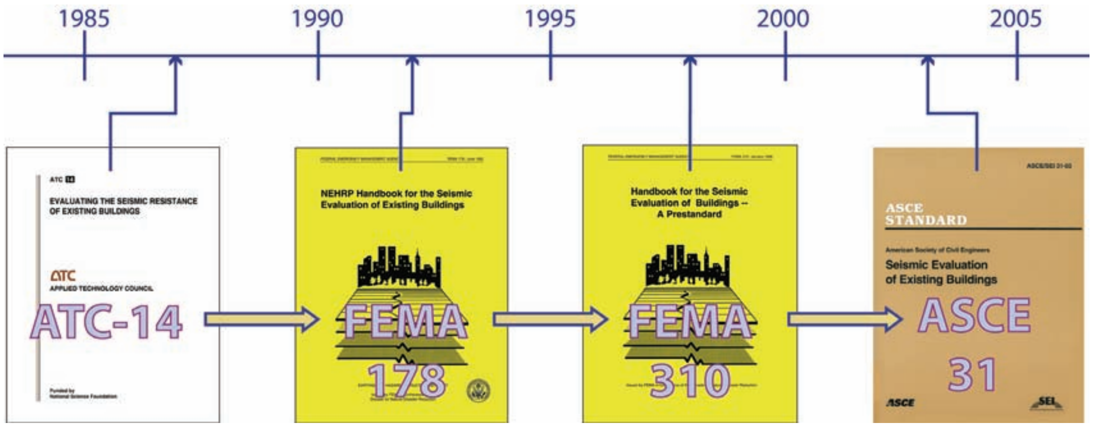

Figure 2-1 Evolution of the development of ASCE/SEI 31

ASCE/SEI 31 defines a three-tiered process in which each successive tier involves more detailed evaluation and increased engineering effort. The additional effort in each tier is intended to achieve greater confidence in the identification and confirmation of seismic deficiencies. The ASCE/SEI 31 evaluation procedure comprises three phases:

- *Screening Phase (Tier 1)*. The basis of the methodology is a checklist procedure utilizing a series of checklists to identify building characteristics that have exhibited poor performance in past earthquakes. Checklists include the basic and supplemental structural checklists, the basic, intermediate, and supplemental nonstructural checklists, and the geologic site hazard and foundation checklists. Selection of appropriate checklists depends on the common building type designation, level of seismicity, and desired level of performance. The checklists contain statements that are used to define the scope of the evaluation and identify potential deficiencies that can be investigated in more detail.
- *Evaluation Phase (Tier 2)*. If a building does not comply with one or more checklist statements in Tier 1, the condition can investigated further to confirm or eliminate the deficiency. The Tier 2 Evaluation Phase is conducted using linear static or linear dynamic force-based calculations on a deficiency-only or full-building basis.
- *Detailed Evaluation Phase (Tier 3)*. If deficiencies are not eliminated in the Tier 2 Evaluation Phase, they can be investigated further using nonlinear static or nonlinear dynamic analyses. The Tier 3 Detailed Evaluation Phase is based on the procedures and criteria contained in ASCE/SEI 41, although the use of reduced criteria (75% of the specified demand) is permitted for this evaluation.

Engineering effort required for Tier 1 screening is relatively small (on the order of days). Depending upon the number of potential deficiencies, the effort for a Tier 2 evaluation is greater (on the order of weeks). A Tier 3 detailed nonlinear analysis can be very time-consuming (a month or more).

Experience in regions of high seismicity has shown that many pre-1980 concrete buildings require retrofit, or further investigation, as a result of a Tier 2 evaluation. Due to the time and expense associated with a Tier 3 detailed evaluation, and the uncertainty associated with being able to eliminate nonductile concrete deficiencies as potential collapse concerns, many buildings owners proceed directly to retrofit rather than performing a Tier 3 detailed evaluation.

2.4.2 ASCE/SEI 41 Standard for Seismic Rehabilitation of Existing Buildings

ASCE/SEI 41 is a national consensus standard for the seismic rehabilitation of existing buildings. It defines a performance-based approach for seismic analysis and design that can be used to achieve a desired performance objective selected from a range of performance levels (Collapse, Collapse Prevention, Life Safety, Immediate Occupancy, and Operational) at any seismic hazard level. As illustrated in Figure 2-2, the procedures and criteria contained within ASCE/SEI 41 were initially developed in the early 1990s, and are based on a series of predecessor documents dating back to FEMA 273, *NEHRP Guidelines for the Seismic Rehabilitation of Buildings* (FEMA, 1997).

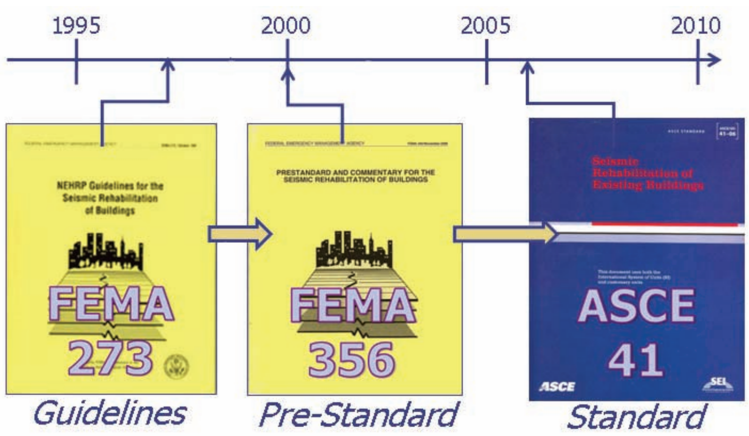

Figure 2-2   Evolution of the development of ASCE/SEI 41

ASCE/SEI 41 is intended to be comprehensive in scope and generally applicable to structural and nonstructural components in buildings of any configuration and any construction type. Engineering analysis is based on a series of linear, nonlinear, static, and dynamic analysis options, each of which involves increasing levels of effort intended to achieve greater confidence in the resulting rehabilitation design.

The performance-based engineering framework involves the estimation of nonlinear deformation demands (calculated directly or through forced-based surrogate procedures), which are then compared to acceptance criteria in the form of acceptable deformation limits that vary with the selected performance level. The terminology for performance levels is identical to ASCE/SEI 31, but the criteria are somewhat different.

Force and deformation acceptability criteria for concrete components are provided in Chapter 6 of ASCE/SEI 41. Modeling parameters and acceptance criteria for concrete columns, slab-column connections, and shear wall components were updated substantially with the release of *Supplement 1* to ASCE/SEI 41 (ASCE, 2007b).

2.4.3 Limitations Relative to Nonductile Concrete Buildings and Needed Improvements

As currently formulated, ASCE/SEI 31 and ASCE/SEI 41 are not capable of reliably determining the relative collapse risk between different nonductile concrete buildings. From a public policy standpoint, the ability to economically make this distinction across an inventory of existing concrete buildings is a critical need.

Modification of ASCE/SEI 31 and ASCE/SEI 41 to differentiate collapse risk in an inventory of non-ductile concrete buildings would need to address the following major limitations:

1. ASCE/SEI 31 checklists cover the most common deficiencies found in concrete buildings. They do not, however, address the relative importance of these deficiencies, or their interaction, with respect to the collapse potential of a specific building. Current model buildings types do not reflect the wide variation in building characteristics or configuration found in existing concrete construction. Analytical studies are needed to investigate how the interaction of multiple deficiencies can affect the collapse potential of a building.
2. Collapse probability is highly dependent on the dominant mechanism of lateral inelastic response. Presently, the dominant mechanism cannot be reliably predicted without nonlinear dynamic analysis. Focused analytical studies are needed to identify building and component parameters that are better indicators of potential collapse mechanisms, leading to more rapid, but still reliable, techniques for assessment.
3. Current procedures are fundamentally deterministic, and the associated degree of uncertainty and reliability are generally not specified. Changes in modeling parameters and acceptance criteria for concrete columns in ASCE/SEI 41 Supplement 1 provide an example where scatter in data and degree of conservatism are explicitly stated. Similar transparency in modeling parameters and acceptance criteria for all concrete components is needed.
4. The lack of a consistent methodology for the selection of modeling parameters and acceptance criteria has led to different levels of conservatism reflected in the limits specified for different concrete components. Using different levels of conservatism in the assessment of different components can result in unreliable predictions of the expected collapse mode or mechanism. A consistent methodology for the selection of modeling parameters and acceptance criteria is

needed to update criteria for all concrete components and improve collapse prediction.

5. Current procedures deem a building deficient if any single component fails its acceptability criteria. For example, strict interpretation of ASCE/SEI 41 leads to unacceptable behavior if a single component loses vertical load-carrying capacity. Seismic performance, particularly collapse, is not so narrowly defined. Most structures have some ability to redistribute load. Realistic assessments must be based on a broader view of the nature and extent of component behavior and the interaction of various components contributing to important global damage states. System capacity must be considered in the development of an improved evaluation process. Collapse simulation studies of building prototype models are needed to identify system response parameters that are more reliable indicators of probable system collapse.

In the program plan recommended herein, it is anticipated that ASCE/SEI 31 could be modified and expanded to address these needs at the screening phase, and further updates to ASCE/SEI 41 modeling parameters and acceptance criteria could enable further distinction of building collapse risks during the detailed evaluation and rehabilitation design phases. Possible approaches for addressing the above limitations are presented in the chapters that follow.

Chapter 3

Summary of NEES Grand Challenge: *Mitigation of Collapse Risks in Older Reinforced Concrete Buildings*

The George E. Brown, Jr. Network for Earthquake Engineering Simulation (NEES) Grand Challenge project entitled *Mitigation of Collapse Risks in Older Reinforced Concrete Buildings* was initiated in 2006. Funded by the National Science Foundation (NSF), this project is focused on understanding the risk associated with collapse of older, West Coast, concrete buildings during earthquakes, and investigating strategies to reduce that risk. Data from this research program is expected to be directly usable in the development of guidance on mitigation of collapse risks in nonductile concrete buildings.

This chapter summarizes the scope and objectives of the NEES Grand Challenge project, and describes details associated with component testing and analytical studies that are directly relevant to program plan recommended herein.

3.1 Overview

The NEES Grand Challenge project was developed under the premise that within a large inventory of older concrete buildings, a relatively small fraction of these would be vulnerable to collapse during strong earthquake shaking, and that collapse triggers could be targeted for investigation in this subset of buildings to reduce retrofit costs, thereby achieving more efficient mitigation than is possible with currently available technologies.

Work on the project is planned to occur over a five-year period ending in December 2011, with total funding of approximately $3.6 million. Research tasks are organized under four themes: (1) exposure; (2) component and system performance; (3) building and regional simulation; and (4) mitigation strategy. Inter-relationships between the themes and tasks are shown in Figure 3-1.

1. *Exposure*. A detailed inventory was developed for a single urban region (City of Los Angeles) to serve as a model for other regions. This inventory, along with other work done in partnership with the Concrete Coalition, provides a snapshot of the older concrete building inventory prevalent in California, and serves as a basis for the development of an inventory methodology. Working in collaboration with the Southern California Earthquake Center, the project has

also developed seismic hazard and ground shaking data for the Los Angeles region.

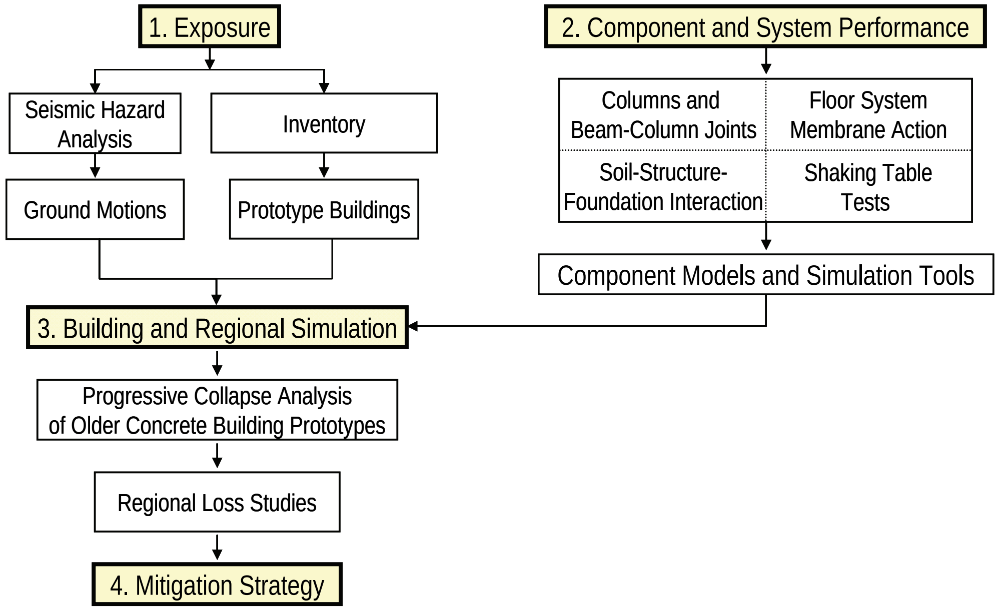

Figure 3-1 Inter-relationships between themes and tasks of the NEES Grand Challenge project.

2. *Component and System Performance*. Laboratory and field experiments are being conducted on components, subassemblies, and soil-foundation-structure systems to better understand conditions that lead to collapse. Laboratory tests funded under this project include tests on columns, corner beam-column joints, and floor systems sustaining column axial failure. Field tests will investigate soil-foundation-structure interaction under large amplitude shaking. Collaborations with Japan and Taiwan have brought additional shake-table test data on structures of varying complexity. Tests serve as a basis for developing analytical models, including models suitable for implementation by structural engineers and models suitable for incorporation in nonlinear simulation software such as OpenSees (Open Systems for Earthquake Engineering Simulation).

3. *Building and Regional Simulation*. Analytical models are being implemented in nonlinear dynamic analysis software. These capabilities will enable the exploration of conditions that lead to collapse. The project will also develop simplified analytical models for use in regional studies of older concrete buildings in the City of Los Angeles.

4. *Mitigation Strategies*. Mitigation strategies will be investigated. Pending available funding, additional laboratory experiments will be performed on columns retrofitted with simple confinement jackets to better understand what can be done to mitigate collapse triggers associated with columns. Appropriate public policy strategies will also be explored.

3.2 Column Testing

Since failure of concrete columns is a significant collapse trigger in older concrete buildings, a major focus of the NEES Grand Challenge project is laboratory testing of concrete columns susceptible to shear and axial failures. The objective of the NEES Grand Challenge column testing program is to fill gaps in available data to further test and validate underlying empirical models and resulting acceptance criteria.

Results of prior laboratory tests and empirical models were analyzed in Elwood et al. (2007), leading to revised column acceptance criteria and modeling parameters in ASCE/SEI 41 *Supplement 1* (ASCE, 2007b). The scope of the NEES Grand Challenge column testing program is shown in Table 3-1. The program includes study of variations in longitudinal reinforcement ratio, transverse reinforcement, aspect ratio (clear height divided by gross cross-sectional dimension), loading protocol, and axial load level.

Table 3-1 NEES Grand Challenge Column Testing Program

ID	Long Reinf. Ratio	Transverse Reinforcement			Aspect Ratio	Loading Protocol[1]	P/f'_cA_g
		Type	Spacing	Ratio			
KU 1	2.5%	A	18”	0.07%	6.44	U3	0.32
KU 2	2.5%	A	18”	0.07%	6.44	U3	0.22
KU 3	3.1%	A	18”	0.07%	6.44	U3	0.62
KU 4	2.5%	B	18”	0.18%	6.44	U6	0.17
PU1	1.5%	A	18”	0.07%	3.22	U3	0.37
PU2	1.5%	A	8”	0.07%	3.22	U3	0.38
PU3	1.5%	A	18”	0.07%	3.22	B7	0.21
PU4	2.5%	A	18”	0.07%	3.22	U3	0.43
PU5	2.5%	A	18”	0.07%	3.22	B3	0.46
PU6	2.5%	B	18”	0.18%	6.44	B3	0.11
PU7	2.5%	B	18”	0.18%	6.44	B2	0.11
PU8	2.5%	B	18”	0.18%	6.44	B2	0.11

[1] U=Uni-directional; B=Bi-directional; #=number of cycles per drift per direction

A total of twelve specimens were tested, each with an 18-inch square cross-section, 8-bar symmetric longitudinal reinforcement configuration, Grade 60 reinforcement, and concrete strength, f'_c, of 3000 psi to 5000 psi. Transverse reinforcement spacing varied from 8 inches to 18 inches, and details were intentionally configured to be out of conformance with ductile detailing requirements in modern seismic provisions and concrete design standards (Figure 3-2).

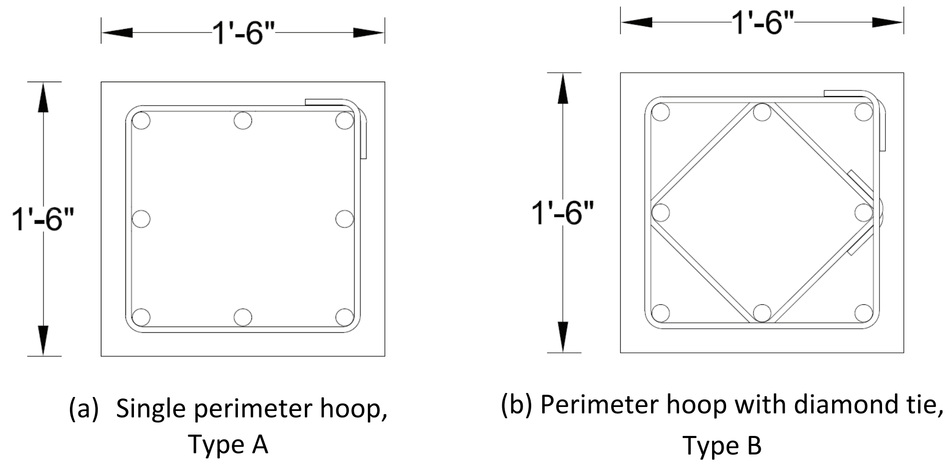

Figure 3-2 Column test specimens.

Specimens were tested in double-curvature (Figure 3-3). The top beam was displaced laterally while rotation in the top and bottom beams was restrained. Axial load was held constant throughout the tests until axial failure was initiated. Specimens were subjected to displacement reversals at increasing amplitudes until the prescribed axial load could no longer be resisted. Some specimens were subjected to displacement reversals in one lateral direction (uni-directional protocol), while others were subjected to displacement reversals in both lateral directions (bi-directional protocol).

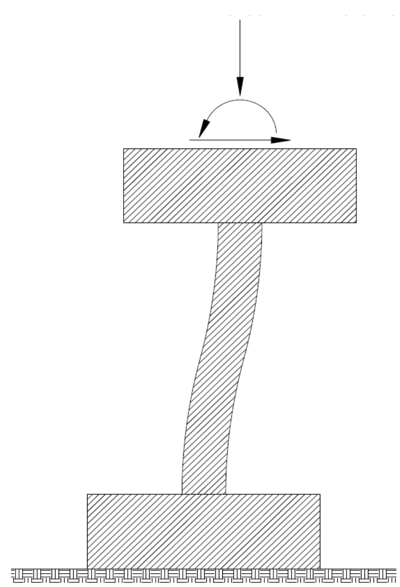

Figure 3-3 Double-curvature column testing configuration.

Figure 3-4 shows the state of one column specimen (PU8) tested to failure. Figure 3-5 plots drift at axial failure versus the axial load and transverse reinforcement quantity for all specimens.

Figure 3-4 Column specimen PU8 tested to axial failure (Courtesy of NEES Grand Challenge).

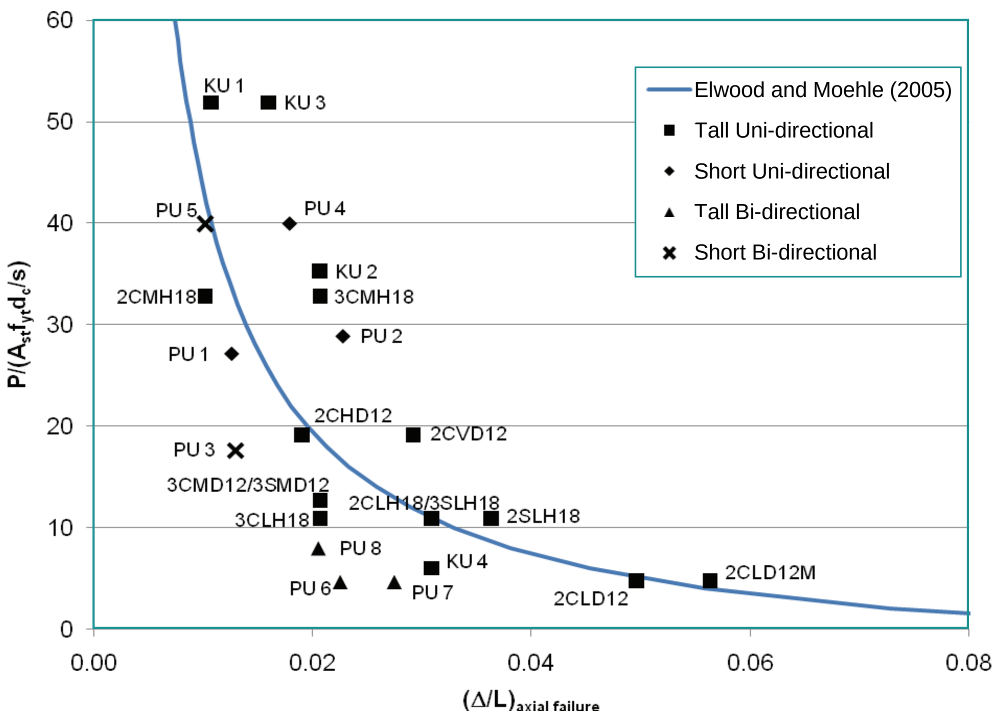

Figure 3-5 Drift at axial failure for all test specimens plotted relative to Elwood and Moehle (2005).

In Figure 3-5, the smooth curve is the relation developed as an estimate of drift capacity based on prior tests (Elwood and Moehle, 2005). The figure shows how the results for NEES Grand Challenge column specimens plot relative to the Elwood and Moehle relation. Results to date from NEES Grand Challenge column testing program indicate that the following changes in test specimen parameters increase the drift at axial failure:

- Decrease in column aspect ratio
- Decrease in axial load level
- Increase in longitudinal reinforcement ratio
- Increase transverse reinforcement ratio
- Decrease in tie size and spacing (with constant transverse reinforcement ratio)
- Decrease in number of displacement cycles

Additionally, it was observed that a uni-directional displacement protocol resulted in larger drifts at axial failure compared to a similar bi-directional displacement protocol. It is expected that, in combination with existing data, supplemental data provided by the NEES Grand Challenge column testing program will serve as a basis for improved acceptance criteria and modeling parameters for non-ductile concrete columns to be developed as part of the program plan recommended herein.

3.3 Beam-Column Joint Testing

Earthquake reconnaissance in the literature includes examples of building collapses that appear to have been caused by damaged beam-column joints. Generally, such failures have been confined to perimeter beam-column connections. Older beam-column joints have been tested previously. These tests have demonstrated weaknesses in some anchorage details, along with a tendency for beam-column joint shear failure to occur under certain conditions.

Complete joint failure, signaled by loss of ability to support column axial loads, however, has seldom been observed in the laboratory. One hypothesis for this observation is that axial forces in previous beam-column joint tests have been lower than occurs in actual buildings, and too low to trigger axial failures. The NEES Grand Challenge project includes a beam-column joint testing program to explore this hypothesis through a series of full-scale tests on corner beam-column joints.

The scope of the NEES Grand Challenge beam-column joint testing program is shown in Table 3-2. The program includes study of variations in joint aspect ratio (ratio of beam depth, h_b, to column depth, h_c), beam reinforcement, column reinforcement, target failure mode, loading protocol, and axial load level.

A total of eight specimens are planned, each with Grade 60 reinforcement, and concrete strength, f'_c, of 3500 psi to 4500 psi. Column longitudinal reinforcement is continuous through the joint, without lap splices, and beam longitudinal reinforcement is continuous across the joint, with standard hooks that extend to the mid-height of the joint. Beam and column transverse reinforcement does not continue into the joint. In some cases the joints are expected to fail before beam yielding (J-Type), and in other cases the joint is expected to fail after beam yielding (BJ-Type).

Table 3-2 NEES Grand Challenge Beam-Column Joint Testing Program

		Beam Reinf.					$P/ f_c' A_g$	
ID	Joint Aspect Ratio	Top	Bottom	Column Reinf.	Target Failure Mode[1]	Loading Protocol[2]	initial	@ shear failure
1	1/1	4 # 6	4 # 6	8 # 8	BJ	U2	0.08	0.12
2	1/1	4 # 8	4 # 7	8 # 9	J	U2	0.15	0.24
3	5/3	4 # 6	4 # 6	8 # 8	BJ	U2	0.10	0.16
4	5/3	4 # 8	4 # 7	8 # 9	J	U2	0.11	0.17
5	1/1	4 # 10	4 # 9	8 # 10	J	U2	0.21	0.31
6	1/1	4 # 10	4 # 9	8 # 10	J	B2	0.21	0.45[3]
7	5/3	4 # 9	4 # 8	8 # 10	J	U2	0.21	0.45
8	1/1	4 # 6	4 # 6	8 # 10	BJ	U2	0.21	0.45[3]

[1] BJ = joint failure after beam yielding; J = joint failure without beam yielding

[2] U=Uni-directional; B=Bi-directional; #=number of cycles per drift per direction

[3] Predicted or target values based on test plan and analytical models

Figure 3-6 shows the general configuration of the beam-column joint test specimens in the loading rig. Specimens were tested first by loading beams and columns to target gravity load levels, then by cycling the beams up and down to simulate lateral drift cycles in the two orthogonal directions. Axial loads varied with beam loading to simulate overturning effects. Target axial loads ranged from tension through $0.45A_g\, f_c'$ in compression. Tests were continued until actuator stroke capacity was reached or axial failure occurred.

Figure 3-7 shows the state of one beam-column joint (specimen ID 5) at the end of testing. As of July 2010, the test program was still under way, with six of eight tests completed. At present, it appears that beam-column joints are showing much less vulnerability to axial collapse than columns. The results should demonstrate the axial collapse fragility of corner beam-column joints. It is expected that, in combination with existing data, supplemental data provided by the NEES Grand Challenge beam-column joint testing program will serve as a basis for improved joint strength modeling parameters and acceptance criteria to be developed as part of the program plan recommended herein.

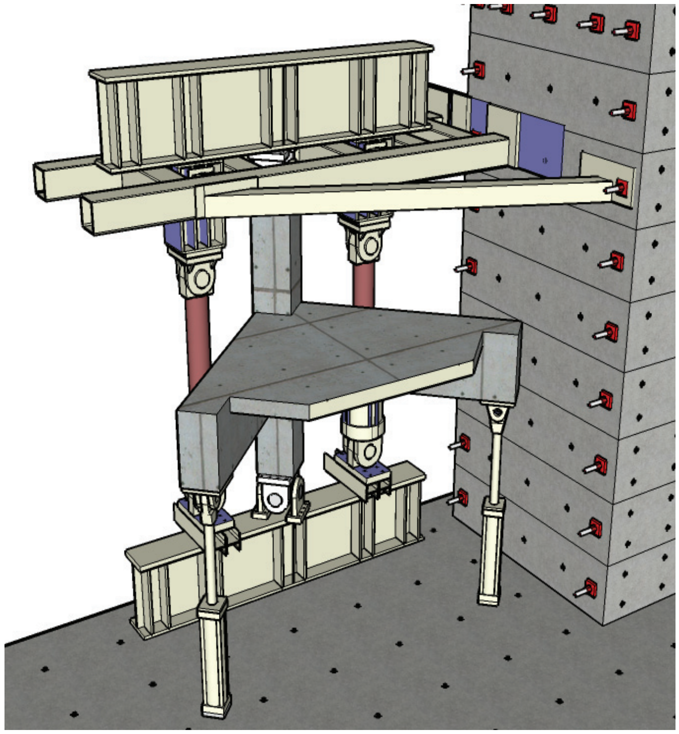

Figure 3-6 Beam-column joint testing configuration (Courtesy of NEES Grand Challenge).

Figure 3-7 Beam-column joint specimen ID 5 tested to failure (Courtesy of NEES Grand Challenge).

3.4 Building Simulation Models

Analytical models of component behavior, including axial collapse models, will be implemented in OpenSees. These models will enable advanced collapse simulations using detailed or simplified analytical models of older concrete buildings. The principal objective of the NEES Grand Challenge building simulation study is the development of collapse fragilities for a limited set of simplified building models.

A building inventory conducted as part of the NEES Grand Challenge project has established the number, age, size, occupancy, and general configuration of older concrete buildings in the City of Los Angeles. In parallel with the inventory development, focus group discussions with practicing structural engineers and surveys of concrete building collapses in past earthquakes have enabled development of a list of critical deficiencies for older concrete buildings. For age and configuration/size categories with large building populations, a series of simplified building models will be developed with various combinations of these critical deficiencies. Simplified models will then be subjected to a series of earthquake ground motions representative of the seismic hazard in the City of Los Angeles to establish building collapse fragility relations. These fragility relations will then serve as the basis of loss estimation studies for the City of Los Angeles.

The scope of the NEES Grand Challenge building simulation study will not enable development of a complete set of building fragilities. It is expected this information, in conjunction with additional analytical studies on more complex and realistic building systems, will be used to develop a broader set of fragilities under the program plan recommended herein.

Chapter 4
Common Deficiencies in Nonductile Concrete Buildings

A list of critical deficiencies contributing to the collapse vulnerability of concrete buildings is shown in Figure 4-1. Each has been found to contribute to collapse or partial collapse of concrete buildings in past earthquakes. The order of deficiencies listed in the figure does not imply a level of importance or frequency. Deficiencies A through D are component deficiencies that can limit the ability of a structure to resist seismic loading without collapse. Deficiencies E through J are system-level deficiencies that, alone or in combination with component deficiencies, can elevate the potential for collapse of a structure during strong ground shaking.

Many older concrete buildings contain one or more of the deficiencies identified in Figure 4-1. While these conditions can lead to collapse, there are many examples of buildings that survive strong shaking without collapse. The challenge is to identify when these deficiencies will lead to building collapse and when they will not.

This chapter describes how these common deficiencies can lead to collapse of a reinforced concrete building, and suggests possible retrofit strategies. Additional information on retrofit strategies can be found in FEMA 547 *Techniques for Seismic Rehabilitation of Existing Buildings* (FEMA, 2006). Chapter 6 builds on this list of deficiencies and recommends comprehensive collapse simulation studies to investigate changes in the probability of collapse. It is envisioned that such studies would be used to determine parameters that better identify conditions and buildings that would be subject to collapse.

4.1 Deficiency A: Shear-Critical Columns

Columns designed with inadequate consideration of shear due to seismic loading will likely have widely spaced transverse reinforcement, and can be vulnerable to shear failure before or after flexural yielding. Captive or short columns, with a low ratio of clear height to gross cross-sectional dimension, are particularly vulnerable to shear failure prior to flexural yielding at the column ends.

Shear failure is a result of the opening of diagonal cracks and degradation of the shear transfer mechanism. Further opening of cracks and movement along the diagonal failure plane can lead to loss of axial load-carrying capacity, as shown in Figure 4-2.

Deficiency A: Shear-critical columns	Deficiency F: Overall weak frames
 Shear and axial failure of columns in a moment frame or gravity frame system.	 Overall deficient system strength and stiffness, leading to inadequacy of an otherwise reasonbably configured building.
Deficiency B: Unconfined beam-column Joints	**Deficiency G: Overturning mechanisms**
 Shear and axial failure of unconfined beam-column joints, particularly corner joints.	 Columns prone to crushing from overturning of discontinuous concrete or masonry infill wall.
Deficiency C: Slab-column connections	**Deficiency H: Severe plan irregularity**
 Punching of slab-column connections under imposed lateral drifts.	 Conditions (including some corner buildings) leading to large torsional-induced demands.
Deficiency D: Splice and connectivity weakness	**Deficiency I: Severe vertical irregularity**
 Inadequate splices in plastic hinge regions and weak connectivity between members.	 Setbacks causing concentration of damage and collapse where stiffness and strength changes. Can also be caused by change in material or seismic-force-resisting-system.
Deficiency E: Weak-story mechanism	**Deficiency J: Pounding**
 Weak-column, strong-beam moment frame or similar system prone to story collapse from failure of weak columns subjected to large lateral deformation demands.	 Collapse caused by pounding of adjacent buildings with different story heights and non-coincident floors.

Figure 4-1 Component and system-level seismic deficiencies found in pre-1980 concrete buildings (based on Moehle, 2007).

Figure 4-2 Column shear and axial failure in the 1999 Koceali (Turkey) Earthquake (Courtesy of NISEE Earthquake Engineering Online Archive).

A column may be able to sustain axial loads after shear failure if the axial load is small and a modest amount of transverse reinforcement has been provided. In the case of high axial loads, crushing of both the flexural compression zone and part of the diagonal strut can lead to immediate loss of axial load capacity if there is inadquate transverse reinforcement.

Columns failing in shear experience a loss of vertical load carrying capacity prior to the development of a side-sway collapse mechanism in the system. As axial capacity is lost, gravity loads must be transferred to neighboring columns, which can lead to a progression of overload, damage, and eventual building collapse.

Past experience suggests that column shear and axial failure is one of the most prevalent causes of collapse in older concrete buildings. The following retrofit strategies can be used to address this deficiency:

- Stiffening of the structural system to prevent columns from experiencing excessive lateral displacements.

- Enhancement of vulnerable columns (e.g., wrapping) to add confinement and protect against shear failure.
- Addition of a supplemental gravity system to support vertical loads in case of column failure.

Stiffening is sometimes preferred in buildings with many vulnerable columns, as it can be less expensive and less disruptive than retrofitting of individual columns. If the number of vulnerable columns is small, then column enhancement can become an economical alternative. The addition of a supplemental gravity system is sometimes used where unusual column configurations result in questionable column deformation capacity, or where it is not feasible to adequately control building displacements.

4.2 Deficiency B: Unconfined Beam-Column Joints

Beam-column joints lacking adequate transverse reinforcement can be vulnerable to shear failure. Given sufficient axial load, unconfined beam-column joints can also experience axial failure as shown in Figure 4-3.

Figure 4-3 Beam-column joint failures in the 1999 Koceali (Turkey) Earthquake (Courtesy of NISEE Earthquake Engineering Online Archive).

Except for longitudinal bars extending from the intersecting beam and column elements, beam-column joints in older concrete frame buildings built prior to 1976 generally did not have reinforcement in the joint region. Beginning in 1976, beam-column joints in seismic-force-resisting frames in regions of high seismicity were required to have transverse reinforcement to protect against shear and axial failures in the joints. In regions of moderate and low seismicity, practice has been to use minimal joint reinforcement, if any at all. Design practice for joints in gravity frames varies considerably, and it is not unusual to find joints without transverse reinforcement, even in modern construction.

Preferred detailing for beam longitudinal reinforcement is to extend the top and bottom bars to the far side of the joint, with hooks bending into the joint. In older concrete frame construction, top bars often have hooks bent upward, and bottom bars have only a short straight anchorage into the joint. This type of detailing affects the failure mode of the joint and the collapse potential of the building. Unfortunately, few tests of beam-column joints have been carried out to axial failure, making assessment of buildings with such details uncertain.

Exterior joints around the building perimeter, especially corner joints, are vulnerable to failure. Interior joints, with beams framing in on all four sides, are less vulnerable due to the confinement provided by the beams. As with shear-critical columns, failure of a joint can result in redistribution of gravity loads to neighboring joints (and columns), and progressive collapse of a building.

Retrofit strategies for beam-column joints include system response modification or local joint enhancement. Since local retrofit of beam-column joints can be challenging due to interference with beams, slabs, and nonstructural components around the joint, the most common retrofit approach is to reduce building drifts to protect the joints from failure.

4.3 Deficiency C: Slab-Column Connections

When subjected to lateral displacement, especially in the presence of large gravity loads, slab-column connections can experience punching shear failure. The absence of continuous bottom bars or post-tensioned strands can lead to collapse of the floor slab, as shown in Figure 4-4, and impact from floors above can collapse the floors below.

Bottom slab reinforcement that is continuous through the column core, or post-tensioned strands that pass over the column, can prevent complete loss of vertical-load-carrying capacity, but the softening of the connection will result in some transfer of gravity loads to adjacent slab-column connections. Redistribution of gravity loads can overload other connections, however, leading to a progression of punching failures throughout the floor level.

Retrofit strategies for slab-column connections include system response modification or local enhancement. Local enhancement strategies include strengthening of the connection with fiber-reinforced polymer (FRP) strips and ties, addition of through-bolts, or installation of vertical-load-carrying collars around the columns to support the slab if punching occurs. System response strategies involve stiffening the structure to limit rotation demands on the slab-column connections.

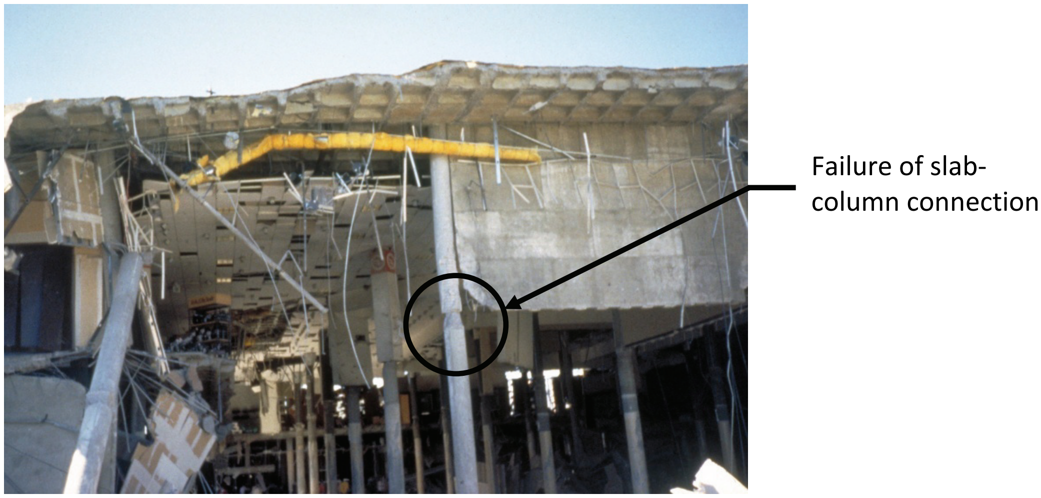

Figure 4-4 Slab-column connection failure in the 1994 Northridge Earthquake (Courtesy of NISEE Earthquake Engineering Online Archive).

4.4 Deficiency D: Splice and Connectivity Weaknesses

Inadequate lap splices located in potential plastic hinge regions, such as at the base of concrete columns, are frequently found in older concrete buildings. Flexural demands at short or unconfined lap splices will result in vertical bond cracks and rapid degradation in flexural capacity. This degradation can also contribute to the formation of a weak-story mechanism in the system (Deficiency E).

Since the damage is generally flexural in nature, only limited diagonal cracking will occur in the splice region. The absence of diagonal cracking suggests that axial loads could be supported at larger drifts as compared to columns experiencing shear failures, although poor confinement and spalling in the splice region would be expected to result in reduced axial load capacity. Accurate assessment of splice strength is critical to understanding the potential collapse mechanism. Underestimation of splice strength can lead to an expectation of flexural-controlled behavior, but the actual splice strength might be sufficient to generate flexural (and corresponding shear) demands to the point that column shear failure could occur.

Other critical connection failures can occur in older concrete buildings. In beams, failures arise due to improper bar splice or cut-off locations, or inadequate anchorage of beam longitudinal reinforcement in beam-column joints. In buildings with long diaphragm spans, inadequate connections between the diaphragms and vertical seismic-force-resisting elements can occur. Failures at multiple locations throughout a building increase the likelihood of collapse due to a loss of member connectivity, as shown in Figure 4-5.

Figure 4-5 Collapse due to connection failures in the 1985 Michoacán (Mexico) Earthquake (Courtesy of Mete Sozen).

Inadequate splices and connections can be addressed by local or global retrofit strategies. Local strategies include added confinement in the splice region, which can be hampered by the presence of connecting members or nonstructural components. If a local retrofit strategy is adopted, increases in flexural strength must not result in a column that is vulnerable to shear failure. Global strategies include reduction of building deformations to limit demands on potentially inadequate splices and connections.

4.5 Deficiency E: Weak-Story Mechanisms

Weak-story mechanisms result in a concentration of inelastic deformation demands in one story of a building. Weak-story mechanisms can occur in buildings with open first stories, and infill or structural walls in upper stories, as shown in Figure 4-6. In such cases, the first story is prone to large drifts, which are exacerbated by P-Delta effects in taller buildings. Weak-story mechanisms can also occur in buildings with deep spandrels (i.e., strong beam-weak column systems) in which a column yielding mechnism is possible at any story. Collapse vulnerability of these buildings is elevated if the columns are also susceptible to shear and axial load failures (Deficiency A).

Weak-story deficiencies can be addressed by local or global retrofit strategies. Column jackets that improve column ductility without adding strength, such as fiber-reinforced polymer wrapping, can improve behavior but do not address the weak-story deficiency. Column jackets and wing walls that increase column strength can be effective in addressing the weak-story deficiency, but can be expensive and disruptive to occupants. Spandrel weakening is sometimes pursued, but is not frequently adopted as a retrofit approach. In many cases, globally improving the strength and stiffness continuity over height of the building is the most effective retrofit strategy. This can be accomplished with the addition of vertical seismic-force-resisting elements, such as shear walls or braced frames.

Figure 4-6 Weak-story mechanism, Olive View Hospital, 1971 San Fernando Earthquake (Courtesy of NISEE Earthquake Engineering Online Archive).

4.6 Deficiency F: Overall Weak Frames

In many older reinforced concrete buildings, a specific seismic-force-resisting system is not present. Instead, frames and infill walls were designed mainly to resist gravity loads, with only nominal lateral resistance for wind loading. In such cases, the overall building lateral strength and stiffness may be very low, leading to excessive story drift and collapse in an earthquake, as shown in Figure 4-7. These buildings can be susceptible to lateral dynamic instability (sidesway collapse), but in most cases loss of vertical-load-carrying capacity will occur first.

Buildings with overall inadequate strength and stiffness must be strengthened, or seismic demands must be reduced through isolation, mass removal, or other response modification techniques. Strength and stiffness can be added with new vertical seismic-force-resisting elements, such as shear walls or braced frames. Local enhancement or strengthening of individual components is not likely to be effective.

Figure 4-7 Weak frame building collapse in the 1999 Koceali (Turkey) Earthquake (Courtesy of NISEE Earthquake Engineering Online Archive).

4.7 Deficiency G: Overturning Mechanisms

Architectural and functional needs for an open first story, or random placement of walls for reasons other than lateral-force resistance, can sometimes lead to discontinuous walls supported on columns. Such columns are subject to large axial loads due to overturning. With or without significant lateral demands, these columns are susceptible to failure due to axial crushing, as shown in Figure 4-8.

Figure 4-8 Column crushing due to discontinuous wall system, 1979 Imperial Valley Earthquake (Courtesy of NISEE Earthquake Engineering Online Archive).

Axial crushing is distinct from a weak-story collapse, which is precipitated by large lateral displacements. Axial crushing can occur even in cases where other walls in the story serve to limit lateral displacement, but where seismic forces in the walls above must transfer into alternative wall lines at the level in question.

A similar condition arises where a shear wall is continuous, but the length is shortened in the first story to accommodate parking or other functional needs. Collapse susceptibility will depend on the degree of discontinuity, detailing, and seismic demands, although this condition is generally less severe than a column-supported wall.

Retrofit of buildings with an overturning deficiency essentially requires continuity in the load path to the base of the structure. This can be achieved by infilling framing bays below discontinuous walls, or by strengthening the supporting columns to increase axial load strength and ductility. In general, it is not sufficient to merely attempt to transfer shear forces out of discontinuous walls to other wall lines, as large overturning axial forces will still be generated in the columns below.

4.8 Deficiency H: Severe Plan Irregularity

Severe plan irregularities can result from an asymmetric building configuration (e.g., T, C, or L-shaped building plans) without seismic joints, or from the asymmetric positioning of stiff lateral force resisting elements. Sever plan irregularities can result in dynamic behavior governed by system torsion, leading to large displacement demands and collapse on the "soft" side of the building, as shown in Figure 4-9.

Figure 4-9 Collapse due to torsional drift demands in the 1999 Athens (Greece) Earthquake (Courtesy of NISEE Earthquake Engineering Online Archive).

If torsionally irregular buildings include other potential deficiencies, such as shear-critical columns, unconfined beam-column joints, slab-column connections, or connection weaknesses, then collapse vulnerability increases. Retrofit should include global reduction of the irregularity through the addition of appropriately located vertical seismic-force-resisting elements, such as shear walls or braced frames. Local retrofit strategies could be adopted to enhance the strength and ductility of selected components, but this would not reduce the torsional demands imposed by the irregularity.

4.9 Deficiency I: Severe Vertical Irregularity

Vertical irregularities can arise from setbacks in the building profile, or from discontinuation of selected vertical elements of the seismic-force-resisting system. Such irregularities can result in excessive displacement and force demands just above the vertical irregularity, as shown in Figure 4-10.

Figure 4-10 Story damage due to vertical irregularity, 2010 Maule (Chile) Earthquake (Courtesy of Jack Moehle).

A similar concentration of demands can occur at the connection between two different structural systems (such as a transition from concrete-encased steel frames to concrete frames, as was observed in the 1995 Kobe Earthquake). If vertically irregular building includes other potential deficiencies (e.g., component deficiencies A through D), a story collapse can occur.

Similar to buildings with plan irregularities, retrofit of buildings with severe vertical irregularities should address the irregularity directly, through global strengthening and stiffening of the weaker story. Care must be taken to avoid creating a weak- or soft-story mechanism in an adjacent story.

4.10 Deficiency J: Pounding

Pounding between two buildings can occur where building separations are inadequate. If the story heights are different, floor slabs are located at different elevations, and the slabs of one building can impact the columns of an adjacent building. Pounding can lead to severe column damage and axial load failure. If the building also includes other potential deficiencies (e.g., component deficiencies A through D), a story collapse can occur.

Pounding problems are especially severe where two buildings of different periods are free to oscillate independently (with the exception of pounding). Where buildings of different heights are adjacent to one another, upper levels in the taller building can experience collapse due to excessive demands caused by dynamic pounding effects, as shown in Figure 4-11. Where buildings are located within in a block of similar height buildings, collapse due to pounding is less likely to occur since differential movement and dynamic effects are constrained by the presence of the adjacent structures.

Figure 4-11 Collapse of upper stories due to building pounding in the 1985 Michoacán (Mexico) Earthquake (Courtesy of Jack Moehle).

Comprehensive retrofit of this deficiency, taking into account the dynamic demands on both buildings, is a technical challenge. It is also a logistical challenge since adjacent buildings are likely to be owned by different parties, and are not likely to be undergoing a seismic retrofit at the same time. A potential retrofit strategy could include providing energy dissipation devices at the levels of anticipated pounding, with struts taking the impact forces down (or up) to the nearest floor diaphragms. Alternatively, one or both buildings could be stiffened to ensure that maximum drifts do not exceed the available clearance between the buildings.

Chapter 5

Recommended Guidance Documents

Based on limitations in current seismic evaluation and rehabilitation practice in the United States (Chapter 2), a review of information currently being developed in the NEES Grand Challenge project *Mitigation of Collapse Risks in Older Reinforced Concrete Buildings* (Chapter 3), and an understanding of common deficiencies found in nonductile concrete buildings (Chapter 4), the following critical needs for addressing the collapse risk associated with older concrete construction have been identified:

- Improved procedures for identifying building systems vulnerable to collapse, including simple tools that do not require detailed analysis.
- Updated acceptance criteria for concrete components based on latest research results.
- Identification of cost-effective mitigation strategies to reduce collapse risk in existing concrete buildings.

To address these needs, the development of a series of guidance documents is recommended. This chapter describes the organization and content of these documents. Draft outlines of selected documents are provided in the appendices.

5.1 Guidance for Collapse Assessment and Mitigation Strategies for Existing Reinforced Concrete Buildings

Under the umbrella title *Guidance for Collapse Assessment and Mitigation Strategies for Existing Reinforced Concrete Buildings,* a series of guidance documents is recommended to support the future development of collapse assessment and mitigation strategies for a range of concrete components and building types. Currently, the following eight documents are envisioned; however, other documents could be conceived in the future to extend the series and address future developing needs:

1. Assessment of Collapse Potential and Mitigation Strategies
2. Acceptance Criteria and Modeling Parameters for Concrete Components: Columns
3. Acceptance Criteria and Modeling Parameters for Concrete Components: Beam-Column Joints

4. Acceptance Criteria and Modeling Parameters for Concrete Components: Slab-Column Systems
5. Acceptance Criteria and Modeling Parameters for Concrete Components: Walls
6. Acceptance Criteria and Modeling Parameters for Concrete Components: Infill Frames
7. Acceptance Criteria and Modeling Parameters for Concrete Components: Beams
8. Acceptance Criteria and Modeling Parameters for Concrete Components: Rehabilitated Components

The first document is intended to focus on building system behaviour, and the remaining documents focus on individual concrete components. To facilitate early progress toward addressing critical needs, and allow sufficient time for developmental work on longer-term products, the series has been conceived in such a way that individual documents can be released as each product is developed. This will also allow greater flexibility in funding initial product development, and make future updating of the documents easier.

5.2 Assessment of Collapse Potential and Mitigation Strategies

The first document, subtitled *Assessment of Collapse Potential and Mitigation Strategies,* will focus on the need to develop improved procedures for identifying building systems vulnerable to collapse, including simple tools not requiring detailed analyses. This document will also address the need to identify cost-effective mitigation strategies to reduce collapse risk in existing concrete buildings. A draft outline for this document is provided in Appendix A.

It is envisioned that this document will build on the component and system deficiencies identified in Chapter 4. Based on the results of comprehensive collapse simulations, it will identify parameters influencing the collapse vulnerability of concrete buildings, and recommend limits for such parameters for assessing collapse potential. A possible methodology for establishing and identifying parameters, and their associated limits, is described in Chapter 6.

Considering the pressing need for guidance on the identification of collapse-vulnerable nonductile concrete buildings, this first document should be considered a high priority. However, development is expected to be time consuming and the necessary analytical studies will be somewhat dependent on the follow-up development of component modeling and acceptance criteria. A phased approach for the development of this document is described in detail in Chapter 8.

5.3 Acceptance Criteria and Modeling Parameters for Concrete Components

The remaining documents, subtitled *Acceptance Criteria and Modeling Parameters for Concrete Components*, will focus on the need to update criteria for concrete components based on the latest available research results. Each will address acceptance criteria and modeling parameters for individual concrete components or subassemblies with influence on the collapse behavior of concrete buildings.

It is important that the recommended acceptance criteria and modeling parameters have a consistent basis and are determined using a consistent methodology. One such methodology is described in Chapter 7. As currently envisioned, recommendations will be presented in a format that facilitates transfer of results into ASCE/SEI 41 *Seismic Rehabilitation of Existing Buildings*.

Component documents will be developed using currently available test data, along with any additional data that might be available at the time of their development, but this does not imply that our state of knowledge is complete or that testing of concrete components will no longer be needed. It must be emphasized that testing is still necessary to refine our understanding of concrete component behavior to the point of collapse. Criteria provided in the resulting component documents would still benefit from additional testing of concrete components to extreme limits of response, particularly to loss of vertical-load-carrying capacity. Each document will likely include recommendations for additional testing that is needed to fill gaps in available knowledge or enhance criteria with more information.

5.3.1 Columns

Considering their critical role in the collapse behavior of most concrete buildings, columns have been identified as the focus for the first component document. Based on a database of column tests collected by the NEES Grand Challenge project, and on supplemental tests performed as part of their column testing program, it is anticipated that improved acceptance criteria and modeling parameters can be readily developed with currently available information. A draft outline for this document is provided in Appendix B.

5.3.2 Beam-Column Joints

Based on a database of beam-column joint tests collected by the NEES Grand Challenge project, and on supplemental tests performed as part of their beam-column joint testing program, it is anticipated that improved acceptance criteria and modeling parameters can be readily developed with currently available information. A draft outline for this document is provided in Appendix C.

5.3.3 Slab-Column Systems

Existing slab-column systems subjected to lateral deformation demands can experience punching shear failures, potentially leading to loss of vertical-load-carrying capacity and progressive collapse. Acceptance criteria and modeling parameters for slab-column systems can be developed, in part, based on an extensive database of slab-column tests collected by Kang and Wallace (2006).

5.3.4 Walls

Concrete wall components frequently control displacement demands in concrete buildings, and proper modeling of walls is critical for the prediction of seismic demands on other building components. Acceptance criteria and modeling parameters for nonductile wall components can be developed, in part, based on data collected by Elwood et al. (2007) in the development of ASCE/SEI 41 *Supplement 1*, and a database of squat wall tests (Gulec et al., 2008).

5.3.5 Infill Frames

Infills significantly influence the seismic behavior of concrete frame buildings. Resources for the development of acceptance criteria and modeling parameters for infill frames includes an ongoing NEES project that is focused on the seismic performance of infill walls in nonductile concrete buildings (Shing et al., 2009).

5.3.6 Beams

Beams are generally not considered critical in controlling the collapse behavior of concrete buildings; however, acceptance criteria and modeling parameters must still be evaluated to properly characterize concrete system behavior. Very limited testing has been conducted on beams with the type of detailing present in older concrete construction. Development of acceptance criteria and modeling parameters for beams will likely require additional tests supplemented by numerical modeling.

5.3.7 Rehabilitated Components

ASCE/SEI 41 does not currently provide modeling parameters and acceptance criteria for rehabilitated components (e.g. fiber-reinforced polymer wrapped columns). Data necessary to develop acceptance criteria and modeling parameters for rehabilitated components should be coordinated with the efforts of ACI Committee 369 on Seismic Repair and Rehabilitation, and include data for rehabilitated concrete columns (Brena and Alcocer, 2009).

Chapter 6
Methodology for Assessment of Collapse Indicators

Development of the first recommended product, *Assessment of Collapse Potential and Mitigation Strategies*, will require focused analytical study to establish limits on design and response parameters that are correlated with the collapse probability of nonductile concrete buildings. It is envisioned that these parameters, termed *collapse indicators*, are related to the critical deficiencies described in Chapter 4, and will be used to more readily identify buildings with an elevated probability of collapse.

This chapter describes a methodology for establishing and assessing collapse indicators for nonductile concrete buildings. It should be emphasized that the initial list of collapse indicators, and the proposed analytical studies, should be viewed as preliminary, and will need to be confirmed as the recommended studies proceed.

6.1 Preliminary List of Potential Collapse Indicators

Ideally, there should be a spectrum of collapse indicators, ranging from those appropriate for rapid assessment to those appropriate for detailed assessment. Collapse indicators for rapid assessment should be simple parameters that can be established from basic information that is available from a survey of the building or review of engineering drawings. Collapse indicators for detailed assessment can make use of results from nonlinear analyses.

Table 6-1 provides a list of potential collapse indicators. They have been identified as component or system-level parameters, and categorized as follows:

- *Design parameter collapse indicators*: Determined based on building design features including reinforcement details, structural system layout, and relative strength and stiffness of members. These can be further sub-categorized as *rapid assessment*, which can be determined from a survey of the building or review of engineering drawings, or *engineering calculation*, which require some calculation of capacities and demands, but not detailed nonlinear analysis.
- *Response parameter collapse indicators*: Determined through analysis of the nonlinear response of a structure.

Where a collapse indicator in Table 6-1 is related to one or more of the common building deficiencies listed in Chapter 4, the applicable deficiencies have been identified in parentheses.

Table 6-1 Potential Collapse Indicators

	Collapse Indicator[1]		System-level		Component-level
Design Parameters	Rapid Assessment (*RA*) *Quantities that can be determined from a survey of the building or review of engineering drawings.*	*RA-S1.*	Maximum ratio of column-to-floor area ratios for two adjacent stories (Deficiencies E and G).	*RA-C1.*	Average minimum column transverse reinforcement ratio for each story (Deficiency A).
		RA-S2.	Maximum ratio of horizontal dimension of the SFRS in adjacent stories (Deficiencies E and I).	*RA-C2.*	Minimum column aspect ratio (Deficiency A)
		RA-S3.	Maximum ratio of in-plane offset of SFRS from one story to the next to the in-plane dimension of the SFRS (Deficiency I)	*RA-C3.*	Misalignment of stories in adjacent buildings (Deficiency J)
		RA-S4.	Plan configuration (L or T shape versus rectangular) (Deficiency H)		
		RA-S5.	Minimum ratio of column area to wall area at each story[2]		
	Engineering Calculations (*EC*) *Quantities that require some calculation of capacities and demands based on engineering drawings, but do not require results from nonlinear building analyses.*	*EC-S1.*	Maximum ratio of story stiffness for two adjacent stories	*EC-C1.*	Maximum ratio of plastic shear capacity ($2M_p/L$) to column shear strength, V_p/V_n (Deficiency A).
		EC-S2.	Maximum ratio of story shear strength for two adjacent stories (Deficiency E)	*EC-C2.*	Maximum axial load ratio for columns with $V_p/V_n > 0.7$ (Deficiency A)
		EC-S3.	Maximum ratio of eccentricity (distance from center of mass to center of rigidity or center of strength) to the dimension of the building perpendicular to the direction of motion (Deficiency H).	*EC-C3.*	Maximum ratio of axial load to strength of transverse reinforcement (45 deg truss model) (Deficiency A)
	.	*EC-S4.*	Portion of story gravity loads supported by columns with ratio of plastic shear demand to shear capacity > 0.7 (Deficiency A)	*EC-C4.*	Maximum ratio of joint shear demand (from column bar force at yield) to joint shear capacity for exterior joints (Deficiency B)
				EC-C5.	Maximum gravity shear ratio on slab-column connections (Deficiency C)
Response Parameters	Building Analysis (*BA*) *Quantities based on results from nonlinear building analyses.*	*BA-S1.*	Maximum degradation in base or story shear resistance (Deficiencies A-B,D-I)	*BA-C1.*	Maximum drift ratio (Deficiencies A-F, H-I)
		BA-S2.	Maximum fraction of columns at a story experiencing shear failures (Deficiencies A, H-I)	*BA-C2.*	Maximum ratio of deformation demands to ASCE/SEI 41 limits for columns, joints, slab-column connections and walls (Deficiencies A-I)
		BA-S3.	Maximum fraction of columns at a story experiencing axial failures (Deficiencies A, H-I)		
		BA-S4.	Minimum strength ratio (as defined in ASCE/SEI 41) (Deficiency F)		
Other		*O-S1.*	Weak soils likely to result in overturning or large deformation demands in the building.		

[1] Collapse indicator notation: *RA* = Rapid Assessment; *EC* = Engineering Calculation; *BA* = Building Analysis; *O* = Other; *S* = System; *C* = Component.

[2] May not result in collapse but could help prevent collapse if a mechanism forms.

It is anticipated that relationships are likely to exist among the different categories of collapse indicators, and that assessment could be hierarchical. For example, if the risk of collapse cannot be eliminated based on rapid assessment with design parameter indicators alone, then detailed assessment would be performed using more detailed analyses and response parameter indicators. Also, vector combinations of multiple collapse indicators could be found to provide a better indication of collapse potential. One such example might be minimum transverse reinforcement ratio (*RA-C1*) less than a specified value, in combination with column floor area ratio (*RA-S1*) greater than a specified value, correlating to higher collapse potential.

Design parameter indicators and system-level response parameter indicators will be addressed in the first document, *Assessment of Collapse Potential and Mitigation Strategies*. Component-level Building Analysis indicators (*BA-C1* and *BA-C2*) can be interpreted as equivalent to component acceptance criteria found in ASCE/SEI 41, and will be addressed in the series of component documents, *Acceptance Criteria and Modeling Parameters for Concrete Components*.

6.2 Focused Analytical Studies

Focused analytical studies are necessary to establish a correlation between building design and response parameters and the probability of collapse. In order to identify appropriate and reliable collapse indicators, analytical models using research oriented structural analysis software (e.g., *OpenSees* – Open Systems for Earthquake Engineering Simulation) are needed. Such models would be used to identify quantitative limits on collapse indicators that have strong correlation with collapse potential.

As currently envisioned, focused analytical studies would be performed on two types of models: (1) simplified models; and (2) building prototype models. Using these models, building characteristics (e.g., dimensions, geometry, and mass) can be varied parametrically to explore effects on building response and collapse probability.

6.2.1 Simplified Models

A subset of system-level collapse indicators can be investigated using simplified nonlinear models. Simplified models are primarily intended to observe trends in building response quantities given variations in selected collapse indicators, and will not necessarily capture loss of vertical-load-carrying capacity.

Results from simplified analyses, however, could be used to approximate probabilities of collapse by comparing the predicted drift demands with available models for loss of vertical-load-carrying capacity of critical members (e.g. Elwood and Moehle, 2005). This approach would provide an estimate of the expected changes in the probability of collapse with changes in collapse indicators.

Simplified models could reasonably be used to investigate system-level Engineering Calculation collapse indicators, such as story stiffness ratio (*EC-S1*), story shear strength ratio (*EC-S2*), torsional eccentricity ratio (*EC-S3*). Simplified models could also potentially be used to investigate Rapid Assessment indicators such as column-to-floor-area ratio (*RA-S1*), horizontal system dimension ratio (*RA-S2*), in-plane offset ratio (*RA-S3*), and plan configuration (*RA-S4*).

Figure 6-1 illustrates a simplified model that could be used to investigate the story stiffness ratio (*EC-S1*) and story shear strength ratio (*EC-S2*) for buildings with soft or weak stories. This model could also be modified to investigate the horizontal system dimension ratio (*RA-S2*) for buildings with vertical irregularities (Deficiency I). In this model, the stiffness and strength of each story is represented by a single nonlinear shear spring and is selected based on typical values for real buildings. The parameter *EC-S1* would be represented by the ratio K_1/K_2, and the parameter *EC-S2* would be represented by the ratio V_{y1}/V_{y2}.

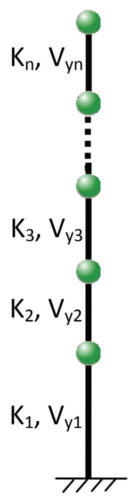

Figure 6-1 Simplified model to investigate collapse indicators based on parameters that vary between stories.

The story stiffness can be varied as a function of the beam-to-column stiffness ratio, which would allow for consideration of lateral system behaviors ranging from cantilever walls to shear frames. Variations in stiffness and strength in each story could be evaluated by simple variations on the same model.

Figure 6-2 illustrates a simplified model that could be used to investigate the torsional eccentricity ratio (*EC-S3*) for buildings with severe plan irregularities (Deficiency H). In the figure, *CM* represents the center of mass, and *CR* represents the initial (elastic) center of rigidity. In such a model, the wall element would represent the primary lateral resistance (K_w, V_{yw}) and torsional resistance (K_θ, T_y) in the building, and the frame element would represent gravity framing located at a maximum distance 'D' from the primary lateral elements. This type of model could be used to analyze a frame system with eccentric core walls around elevators or stairs. The stiffness and strength of the wall and frame elements could be selected based on typical values for real buildings, and the initial ratio (e/D) varied to investigate changes in response.

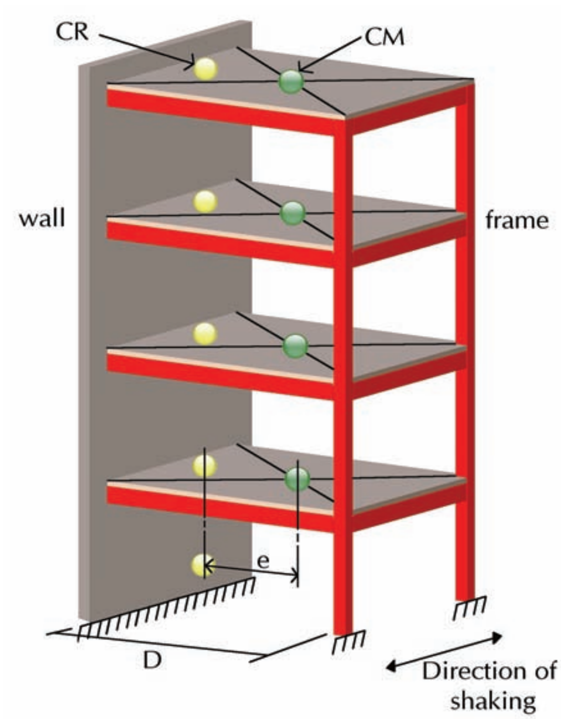

Figure 6-2 Simplified model to investigate collapse indicators based on parameters that vary in plan.

Procedures described in the FEMA P-440A report, *Effects of Strength and Stiffness Degradation on Seismic Response* (FEMA, 2009b), could be used to supplement and expand simplified analysis results. Equivalent single-degree-of-freedom models, developed based on the shape of a building pushover curve, could be used to estimate the range of displacement demands expected at a specific story for different ground motions using incremental dynamic analyses or the *Static Pushover 2 Incremental Dynamic Analysis* open source software tool, SPO2IDA (Vamvatsikos and Cornell, 2006). Comparisons with a cumulative distribution function for drift capacity at loss of vertical-load-carrying capacity can be used to provide an estimate of the probability of collapse, and help identify limits on system-level collapse indicators.

6.2.2 Building Prototype Models

Building prototype models are full building nonlinear models developed from actual structures to explore parametric variations on building characteristics and their effects on response. Building prototype models allow explicit consideration of collapse probability considering loss of vertical-load-carrying capacity, lateral dynamic instability, modeling uncertainty, and ground motion record-to-record variability. Since absolute probability of collapse is difficult to determine, the emphasis should be on relative probabilities of collapse, or changes in probability of collapse due to changes in building characteristics.

Collapse of real buildings is highly dependent on the complex behavior and interaction among individual components. Collapse probabilities must be considered for a cross section of building types to ensure the selected collapse indicators are appropriate for a relatively broad range of buildings characteristics.

Based on an inventory of nonductile concrete building in the Los Angeles area, the NEES Grand Challenge project and the Concrete Coalition have collected a library of models of real concrete buildings. Table 6-2 identifies model building types in the Los Angeles building inventory that could be used as potential building prototype models.

Table 6-2 Potential Model Building Types from the Los Angeles Building Inventory

System Description	Number of Stories[1,2,3]		
	4-8	10-12	14-18
Weak/soft-story with infill walls	X	CSMIP 24579	n/a
Weak/soft-story (architectural design)	X	CSMIP 24322	X
Irregular in plan (property line torsion)	X	X	n/a
Frame with shear-critical columns (spandrel + gravity frame)	n/a	X	n/a
Frame with shear-critical columns (shear critical throughout)	X	X	X
Frame/wall with shear-critical columns	X	X	X
Residential bearing wall	X	n/a	n/a
Early ductile frame	n/a	n/a	CSMIP 24464

[1] X = building type and size available in Los Angeles building inventory

[2] n/a = not available in Los Angeles building inventory

[3] CSMIP building designation provided, where applicable

Building prototype analyses will build on results from simplified models, but will also be able to investigate the full range of component and system-level collapse indicators listed in Table 6-1. Building prototype analyses will also be used to investigate the impact of combinations of collapse indicators. For example, building prototypes could be modified to investigate the impact on the probability of collapse if a building has a story strength ratio (*EC-S1*) less than the limit implied by the simplified analyses, but also has a large percentage of columns with high axial loads (*EC-C2*).

Nonlinear building prototype models used in this study will need to incorporate elements capable of approximating loss of vertical-load-carrying capacity for critical gravity-load supporting components, such as columns (Elwood, 2004) and slab-column connections (Kang et al., 2009), and must account for P-Delta effects. Results from ongoing research within the NEES Grand Challenge project will be necessary to develop models for beam-column joints, and application of element removal methods (Talaat and Mosalam, 2009) should also be explored.

One significant challenge that must be overcome is the distinction between gravity-load collapse and non-convergence due to numerical instability in the model. As envisioned in this study, collapse will be detected based on a comparison of floor-level gravity load demands and capacities (adjusted at each time step to account for member damage and load redistribution). Gravity collapse will be defined as the point at which vertical load demand exceeds the total vertical load capacity at a given floor, and non-convergence of the analysis prior to significant degradation in the capacity to resist gravity loads will not necessarily be considered as collapse. This definition of collapse will need to be confirmed as building prototypes are developed and analyses are conducted.

Two alternative methods for selection of design parameter collapse indicators are described below. One approach is illustrated in Figure 6-3. In this approach, limits are selected based on the relative changes in the collapse fragilities with respect to changes in the collapse indicator parameter.

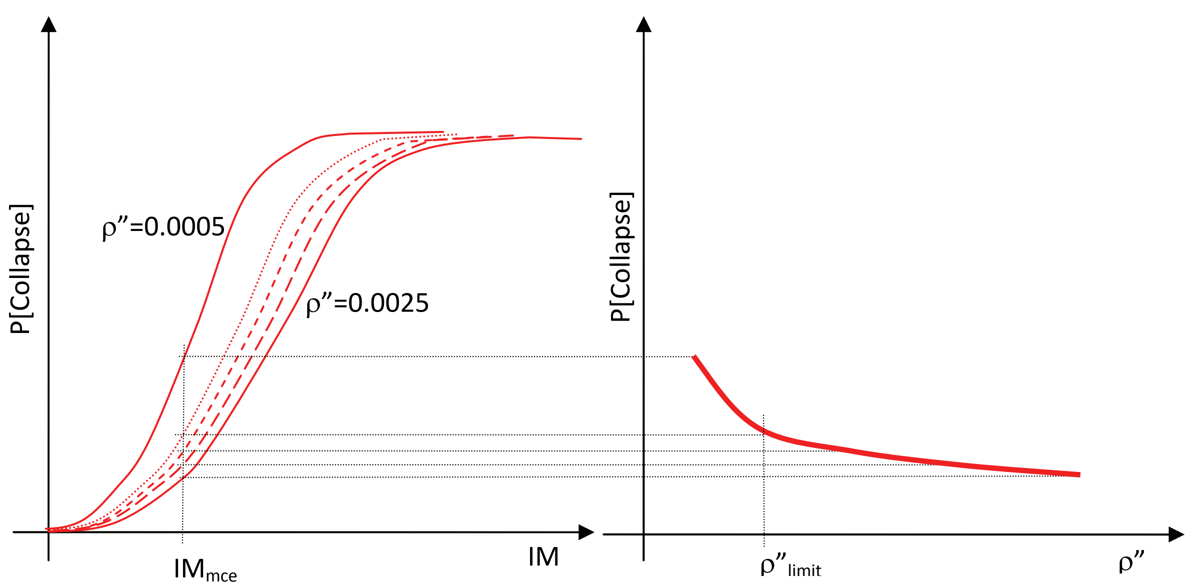

Figure 6-3 Approach for establishing collapse indicator limits based on the relative changes in the collapse fragilities with respect to changes in the collapse indicator parameter (ρ'' = transverse reinforcement ratio; IM = Intensity Measure).

Figure 6-3 shows example collapse fragilities (conjectured) for changes in a selected collapse indicator (e.g., average column transverse reinforcement ratio, *RA-C1*). The curves in the figure suggest that once the transverse reinforcement ratio decreases below about 0.001, the probability of collapse increases rapidly. In this example, 0.001 could be selected as an appropriate limit for this collapse indicator. This assessment would be repeated for several different building types and different hazard levels, and the resulting limits would be compared. An ideal collapse indicator would have only limited variation in the limits suggested by different building types.

A second approach for selection of design parameter collapse indicators is illustrated in Figure 6-4. In this approach, collapse probabilities for prototype buildings are compared with the collapse probability associated with a "good" existing building (i.e., a building for which seismic rehabilitation is not required to achieve a collapse prevention performance level). Prototype buildings would be evaluated considering a range of collapse indicator values. Appropriate limits on the collapse indicators would be determined by the cases where the probability of collapse exceeded that of the benchmark building.

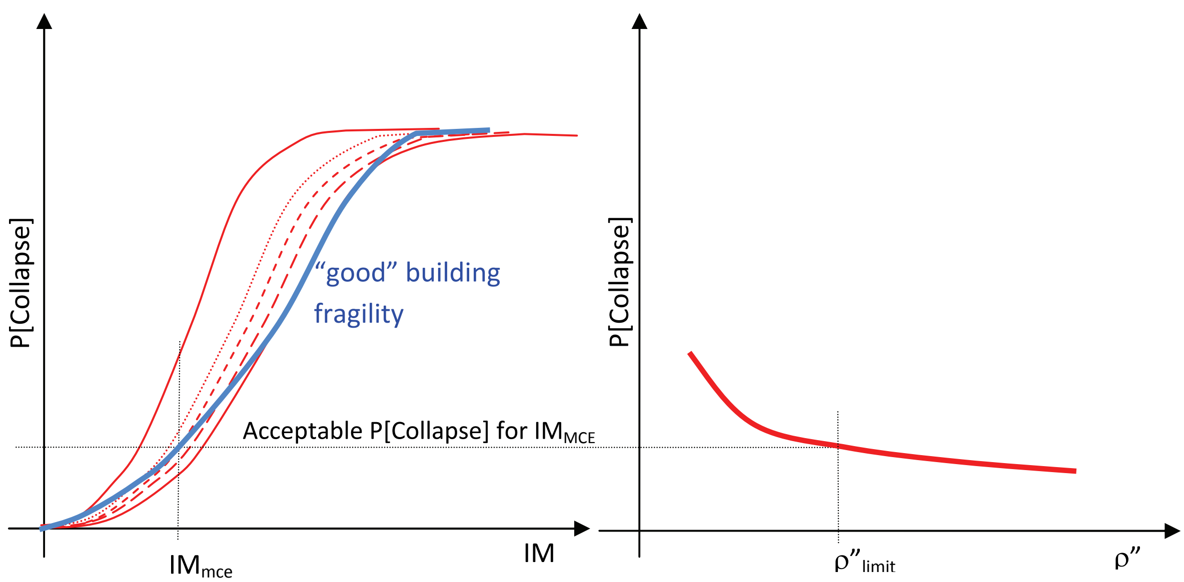

Figure 6-4 Approach for establishing collapse indicator limits based on comparison with a benchmark building collapse fragility (ρ″ = transverse reinforcement ratio; IM = Intensity Measure).

For response parameter collapse indicators, the envisioned process would be similar. Collapse fragilities for building prototypes would be developed and related to a selected system-level response parameter. Similar collapse fragilities would be determined for different building prototypes, and trends in the probabilities of collapse would be compared. Potential limits for the response parameter collapse indicator would be estimated based on a comparison with the collapse fragility for a selected “good” building (similar to Figure 6-4), or based on the point at which the probability of collapse is found to increase rapidly for most building prototypes.

As implemented in a performance assessment, response parameter collapse indicator limits would be compared with responses determined from nonlinear analysis of a building, while design parameter collapse indicator limits would be compared with the relevant design features of a building. Since assessment using design parameter indicators will not directly consider the seismic response of the building in question, it is expected that greater computational effort (i.e., more building prototypes) will be needed to develop reliable design parameter collapse indicators than will be needed to develop response parameter collapse indicators.

Chapter 7

Methodology for Selection of Acceptance Criteria and Modeling Parameters

The recommended series of component documents, *Acceptance Criteria and Modeling Parameters for Concrete Components*, will update acceptance criteria and modeling parameters for concrete components based on recent and future available research information. As currently envisioned, these criteria would supplement or replace ASCE/SEI 41 criteria related to assessment and rehabilitation of concrete buildings.

To properly assess building response and performance, and achieve cost-effective retrofit strategies, modeling parameters must adequately reflect the hysteretic and degrading behavior of concrete components, and acceptance criteria must provide an appropriate degree of conservatism. A consistent statistical basis for selection of acceptance criteria and modeling parameters is needed.

This chapter reviews current ASCE/SEI 41 definitions for acceptance criteria and modeling parameters, and describes a methodology for the selection of improved values for these parameters. It is expected that this methodology will be reviewed and refined during development of the initial component documents.

7.1 Current ASCE/SEI 41 Acceptance Criteria and Modeling Parameters

Acceptance criteria and modeling parameters in ASCE/SEI 41 have generally been selected based on limited data and engineering judgment. A consistent methodology for the interpretation of test data or degree of conservatism has generally not been used across all components, and uncertainty in the prediction of deformation capacities has generally not been accounted for.

The basis for ASCE/SEI 41 acceptance criteria and modeling parameters for primary and secondary concrete components is illustrated in Figure 7-1. In the figure, component stiffness and strength define point B (yield), while modeling parameters define point C (onset of degradation) and point E (loss of residual strength). The slope from C to D is not clearly defined, but generally assumed to be steep. Acceptance criteria for the Collapse Prevention (CP) Performance Level are defined as the deformation (or deformation ratio) at point C for Primary Components, and point E for Secondary Components. Acceptance criteria for the Life Safety (LS) Performance Level are defined as 75% of the acceptance criteria for the Collapse Prevention Performance Level.

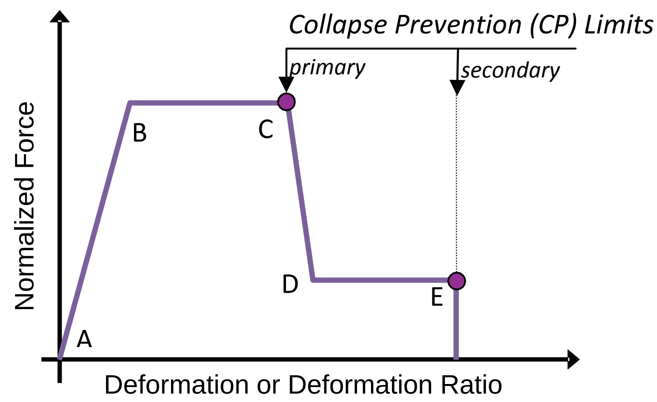

Figure 7-1 Basis for collapse prevention acceptance criteria and modeling parameter limits (adapted from ASCE, 2007).

7.1.1 Improvements in ASCE/SEI 41 Supplement 1

A notable exception is the updated acceptance criteria and modeling parameters for concrete columns incorporated in ASCE/SEI 41 *Supplement 1* (Elwood et al., 2007). Using a database of concrete column tests, modeling parameters for point C were selected to represent a defined probability of failure. The probability of failure deemed acceptable varied based on the severity and consequences of the failure (e.g., 35% for flexural failures, and 15% for shear failures).

Similarly, modeling parameters for point E were selected to represent a defined probability of failure (15% all columns due to the high consequence of axial failure). Consistent with ASCE/SEI 41, other acceptance criteria were established based on the selected modeling parameters and the relationship illustrated in Figure 7-1.

7.1.2 Current Limitations

While updated column acceptance criteria in ASCE/SEI 41 *Supplement 1* reflect the uncertainty in the predicted performance of concrete columns, and were selected with clear justification for the values, some potential limitations remain:

- Modeling parameters were selected based on conservative estimates (rather than mean or median values) of backbone response. This could result in overly conservative (rather than mean or median) predictions of component deformation demands in the structural analysis.
- Different probabilities of failure were used to select updated modeling parameters for columns, and the method of selection was not consistent with how parameters were selected for other components. This could result in an unreliable prediction of component failure and the building failure mode.

- A conservative estimate of point C (onset of degradation) and steep degrading slope reflected in Figure 7-1 can lead to premature predictions of lateral dynamic instability and over-prediction of drift demands.

A new methodology for selection of acceptance criteria and modeling parameters is needed to address the above issues and improve consistency for all concrete components.

7.2 Recommended Methodology for Selection of Acceptance Criteria and Modeling Parameters

The recommended methodology for selection of acceptance criteria and modeling parameters is based on a methodology being developed by ACI Committee 369 (Sezen et al., 2009). In the envisioned methodology, component fragilities, which express the probability of failure at a given deformation demand, will be developed for all concrete components based on laboratory test data and the procedures established under the ATC-58 Project (ATC, 2009a). Where limited test data are available, it is possible to establish coefficients of variation or modify median values based on judgment or expert opinion.

Once component fragility curves are determined, appropriate failure probabilities must be selected to establish the recommended acceptance criteria and modeling parameters. Figure 7-2 illustrates the selection of acceptance criteria based on uncertainty in component behavior. To achieve an appropriate degree of conservatism, considering the large scatter in experimental data, it is likely that a conservative probability of failure (e.g., 15% or 35%) will be necessary for selection of acceptance criteria. To achieve best-estimate predictions of response, modeling parameters should be based on median values (50% probability of failure).

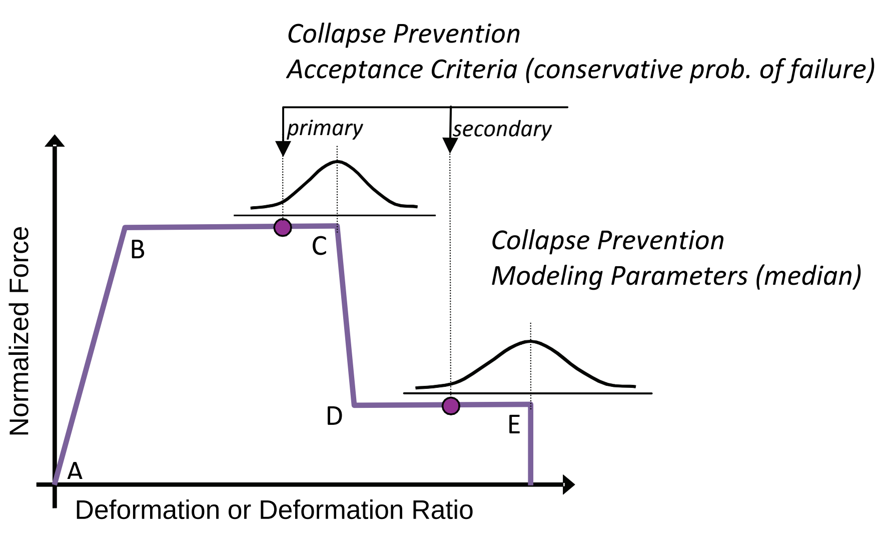

Figure 7-2 Proposed collapse prevention acceptance criteria and modeling parameter limits accounting for uncertainty in component behavior.

With the exception that modeling parameters are based on the median values (50% probability of failure), this approach is consistent with the changes implemented in ASCE/SEI 41 *Supplement 1*. In this approach, conservatism is built into the acceptance criteria, and modeling parameters are set to achieve best estimates of response in the structural analysis.

The degree of conservatism used in the selection of acceptance criteria could depend on many factors. Most importantly, the degree of conservatism should reflect the consequence of failure. When acceptance criteria are selected for a high-consequence failure (e.g., loss of vertical-load-carrying capacity), it is appropriate to consider lower probabilities of failure. The methodology allows for flexibility in considering consequence and uncertainty. It is expected that appropriate levels of conservatism will be selected based on the judgment of a committee, and are likely to be different for different components and performance levels. Complete and transparent documentation of the basis for all updated acceptance criteria and modeling parameters will be necessary.

Chapter 8
Work Plan: Summary of Tasks, Schedule, and Budget

This chapter summarizes the recommended work plan tasks, schedule, and budget for a multi-year program to develop *Guidance for Collapse Assessment and Mitigation Strategies for Existing Reinforced Concrete Buildings*, which comprises the following eight recommended documents:

1. Assessment of Collapse Potential and Mitigation Strategies
2. Acceptance Criteria and Modeling Parameters for Concrete Components: Columns
3. Acceptance Criteria and Modeling Parameters for Concrete Components: Beam-Column Joints
4. Acceptance Criteria and Modeling Parameters for Concrete Components: Slab-Column Systems
5. Acceptance Criteria and Modeling Parameters for Concrete Components: Walls
6. Acceptance Criteria and Modeling Parameters for Concrete Components: Infill Frames
7. Acceptance Criteria and Modeling Parameters for Concrete Components: Beams
8. Acceptance Criteria and Modeling Parameters for Concrete Components: Rehabilitated Components

8.1 Work Plan Objectives

The primary objective of the work plan is the development of the recommended guidance documents. Work has been structured to achieve the following additional objectives:

- *Modular approach*. Recommended products have been split into discrete, stand-alone technical resources. A modular approach allows for flexibility in phasing and funding of developmental work. It also allows key collaborators and partnering agencies to select portions of the work plan for funding and development contributing to the common objective.

- *Phased approach*. Work has been split into phases to identify high-priority products, and those products that are judged to be easier to achieve based on currently available information. A phased approach also allows for the dissemination of useful interim products, while allowing sufficient time for the necessary developmental work on longer-term products.

8.2 Work Plan Overview

Work plan tasks are centered on development of the recommended guidance documents. Document 1, *Assessment of Collapse Potential and Mitigation Strategies*, is focused on assessment of building system collapse behavior and mitigation of system deficiencies. The remaining documents (2 through 8), *Acceptance Criteria and Modeling Parameters for Concrete Components*, are focused on improvement of individual component criteria and behavior models based on the latest available research.

A multi-phase, multi-year effort is needed to complete all eight recommended guidance documents. Because of its importance in the overall process for mitigation of collapse risk in older concrete buildings, Document 1 is recommended as the highest priority. Development of Document 1 has been split into three phases. Due to the work of the NEES Grand Challenge project and availability of data from recent research, component Documents 2 and 3 are recommended for development in a separate phase of work in the near-term. The remaining component Documents 4 through 8 are recommended for development in the final phase of work. Work plan phases are summarized below. Work plan tasks are summarized in Table 8-1.

- Phase 1 – Development of Collapse Indicator Methodology
- Phase 2 – Development of Response Parameter Collapse Indicators
- Phase 3 – Development of Design Parameter Collapse Indicators
- Phase 4 – Development of Initial Component Acceptance Criteria and Modeling Parameters
- Phase 5 – Development of Additional Component Acceptance Criteria and Modeling Parameters

8.3 Description of Tasks for Development of Document 1

Development of Document 1, *Assessment of Collapse Potential and Mitigation Strategies*, is recommended as the highest priority. As currently envisioned, work will be based on the methodology for assessment of collapse indicators described in Chapter 6, and will involve considerable time and effort. As such, the developmental effort for Document 1 has been split into three phases (Phases 1, 2, and 3), with each phase intended to result in an interim or final product that will advance the state of knowledge and practice with regard to assessment and mitigation of collapse risk in nonductile concrete buildings.

Table 8-1 Recommended Work Plan - Summary of Tasks

Phase	Document	No.	Task
1	1	1	Development of Collapse Indicator Methodology
	1	1.1	Identification of critical deficiencies and mitigation strategies
	1	1.2	Selection of building prototypes
	1	1.3	Identification of ground motions and component models for collapse simulation
	1	1.4	Evaluation of methodology for selection of collapse indicators
	1	1.5	Development of implementation plan for collapse indicators in seismic rehabilitation process
	1	1.6	Report on Collapse Indicator Methodology
2	1	2	Development of Response Parameter Collapse Indicators
	1	2.1	Conduct of building prototype analyses for response parameter collapse indicators
	1	2.2	Report on Response Parameter Collapse Indicators
3	1	3	Development of Design Parameter Collapse Indicators
	1	3.1	Conduct of simplified analyses for initial identification of design parameter collapse indicators
	1	3.2	Conduct of building prototype analyses to confirm design parameter collapse indicators
	1	3.3	Report on Design Parameter Collapse Indicators
4	2,3	4	Development of Initial Component Acceptance Criteria and Modeling Parameters
	2	4.1	Selection of column acceptance criteria and modeling parameters
	2	4.2	Report on Acceptance Criteria and Modeling Parameters for Concrete Columns
	3	4.3	Selection of beam-column joint acceptance criteria and modeling parameters
	3	4.4	Report on Acceptance Criteria and Modeling Parameters for Concrete Beam-Column Joints
5	4-8	5	Development of Additional Component Acceptance Criteria and Modeling Parameters
	4-8	5.1	Data collection and database development
	4-8	5.2	Selection of acceptance criteria and modeling parameters
	4-8	5.3	Report on Acceptance Criteria and Modeling Parameters for Additional Concrete Components

8.3.1 Phase 1 – Development of Collapse Indicator Methodology

Phase 1 involves the development of the fundamental collapse indicator methodology. This phase is key to the program. It directly affects the work in Phase 2 and Phase 3, and is related to the work on development of component acceptance criteria in Phase 4 and Phase 5. The collapse indicator methodology is linked to the list of potential collapse indicators, identified in Table 6-1 of Chapter 6. Figure 8-1 illustrates how the remaining phases of work are linked to the identification of collapse indicators in Phase 1.

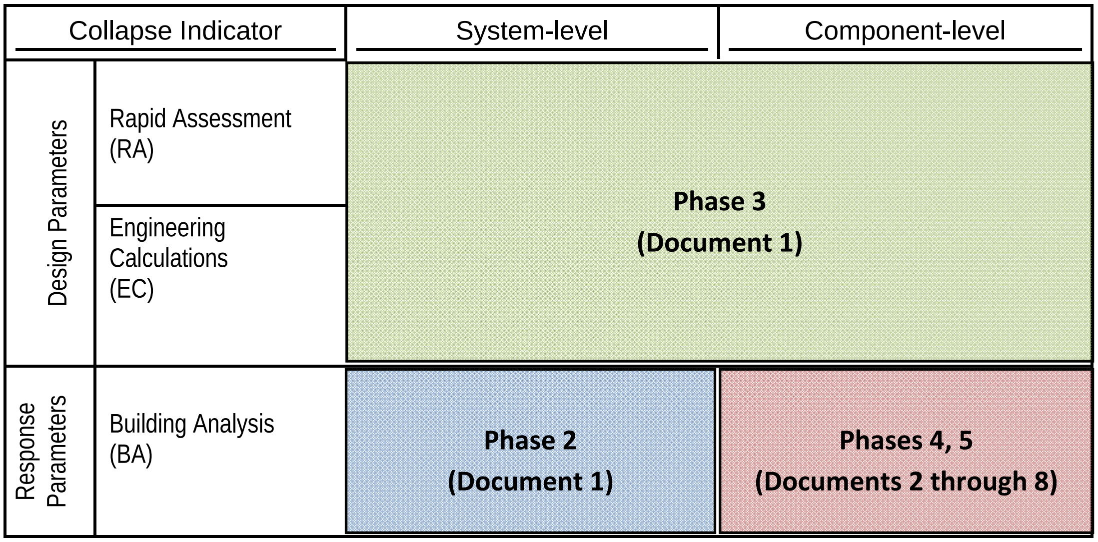

Figure 8-1 Relationship between Phase 1 collapse indicators identified in Table 6-1 and remaining phases of work.

Phase 1 is intended to achieve the following outcomes:

- Identification of critical building systems and deficiencies through a review of concrete building collapses in past earthquakes
- Selection of ground motions for collapse assessment
- Identification of nonlinear component models suitable for collapse simulation
- Selection of prototype buildings from available building inventories
- Development and refinement of the collapse indicator methodology
- Demonstration of the implementation and use of collapse indicators in the assessment and mitigation process

The Phase 1 tasks needed to achieve these outcomes are described below.

Task 1.1: Identification of critical deficiencies and mitigation strategies

This task includes the collection of photographic evidence of concrete building collapses from past earthquakes. This evidence will be used to refine the list of common deficiencies identified in Chapter 4. Appropriate mitigation strategies to achieve a Collapse Prevention performance level for each deficiency will also be identified. An updated list of deficiencies and mitigation strategies will be provided in the Phase 1 report.

Task 1.2: Selection of building prototypes

Selection of building prototypes to be used in analytical studies will need to encompass a broad range of building characteristics and common deficiencies found in available inventories of nonductile concrete buildings. As part of this task, potential collapse indicators will be selected, and building prototypes will need to reflect these characteristics.

Task 1.3: Identification of ground motions and component models for collapse simulation

Before collapse simulation studies can be conducted, appropriate ground motions and nonlinear component models for concrete buildings must be selected. As envisioned, this effort could be accomplished through a combination of an extensive literature review and an invited workshop. The workshop would review the findings of the NEES Grand Challenge project and other relevant research on existing nonlinear analysis models for collapse simulation of concrete buildings. This would include models for concrete columns and beam-column joints, but could also include models for other components deemed critical to the results of collapse simulations (e.g., walls). The workshop would also provide guidance on the modeling approaches and selection of appropriate ground motions to be used in the building prototype studies for the selection of collapse indicators.

Task 1.4: Evaluation of methodology for selection of collapse indicators

The methodology for evaluation of collapse indicators described in Chapter 6 is anticipated to require extensive time and effort. This task involves the conduct of a pilot study to evaluate and refine the proposed methodology, and to help ensure that collapse indicators are identified as expeditiously as possible. This study should incorporate both the simplified analytical studies and the detailed building prototype models.

Task 1.5: Development of implementation plan for collapse indicators in seismic rehabilitation process

As envisioned, design parameter collapse indicators will assess building collapse potential prior to detailed analytical modeling, and response parameter collapse indicators will refine the assessment of collapse potential given the results of detailed analyses. As such, the collapse indicator methodology will provide information previously not used (or available) in the rehabilitation process. This task will develop preliminary recommendations for how collapse indicators could be used during the assessment and rehabilitation process within the context of ASCE/SEI 31 and ASCE/SEI 41. These preliminary recommendations will be reviewed and confirmed after completion of the collapse indicator development in Phase 2 and Phase 3.

Task 1.6: Report on Collapse Indicator Methodology

Outcomes from all Phase 1 tasks on the collapse indicator methodology will be summarized in a Phase 1 report suitable for publication. The purpose of this report will be to: (1) provide the project team with a resource document for subsequent analytical phases of work; and (2) provide the engineering community with important early guidance on factors influencing the collapse of nonductile concrete buildings.

8.3.2 Phase 2 – Development of Response Parameter Collapse Indicators

Response parameter collapse indicators reflect the collapse potential of a building based on the predicted response of the structure through nonlinear analysis. Phase 2 will evaluate system-level response parameter collapse indicators based on a subset of the building prototypes identified in Phase 1. Building prototype models, as described in Chapter 6, will be used to identify limits for response parameter collapse indicators that are found to be correlated with an elevated probability of collapse. Phase 2 tasks for the development of response parameter collapse indicators are described below.

Task 2.1: Conduct of building prototype analyses for response parameter collapse indicators

Prototype buildings will be modeled in nonlinear analysis software capable of detailed collapse simulation (e.g., OpenSees), considering recommended component models and ground motion records identified in Phase 1. Models must include the ability to simulate loss of vertical-load-carrying capacity for critical components. It is assumed that most of this effort will be undertaken by graduate students in synergistic research programs. Prototype buildings will be organized so that parallel efforts can be undertaken by graduate students at multiple universities. These analyses will be used to assess the probability of collapse for prototype buildings. Relative changes in collapse probability will be used to select limits for the associated collapse indicators. Work will require comparisons across multiple prototype buildings to seek out and identify trends in the results.

Task 2.2: Report on Response Parameter Collapse Indicators

Results of building prototype analyses identifying response parameter collapse indicators will be summarized in a Phase 2 report suitable for publication. The purpose of this report will be to: (1) provide the engineering community with information on the probability of collapse for vulnerable nonductile concrete building systems; (2) provide a means to assess collapse using response parameter collapse indicators; and (3) provide updated recommendations on how to implement this information within the context of ASCE/SEI 31 and ASCE/SEI 41.

8.3.3 Phase 3 – Development of Design Parameter Collapse Indicators

Design parameter collapse indicators reflect the collapse potential of a building based on design features (e.g., system configuration, reinforcement details, and relative strength and stiffness of members). Since they do not directly consider the seismic response of the building in question, it is envisioned that more developmental effort will be required to identify and confirm reliable design parameter collapse indicators in Phase 3 than was envisioned for development of response parameter collapse indicators in Phase 2.

Phase 3 will evaluate design parameter collapse indicators based on a the building prototypes identified in Phase 1. Both simplified models and building prototype models, as described in Chapter 6, will be used to identify limits for design parameter collapse indicators that are found to be correlated with an elevated probability of collapse.

Design parameter collapse indicators for both rapid assessment (i.e., quantities that can be determined from a quick survey of the building or review of engineering drawings), and engineering calculation (i.e., quantities that require some calculation based on engineering drawings, but not structural analysis) will be evaluated. Phase 3 tasks for the development of design parameter collapse indicators are described below.

Task 3.1: Conduct of simplified analyses for initial identification of design parameter collapse indicators

Simplified models will be used to evaluate selected design parameter collapse indicators. Results from these analyses will be used to supplement results and refine the scope of building prototype analyses to be conducted under Task 3.2.

Task 3.2: Conduct of building prototype analyses to confirm design parameter collapse indicators

Results from simplified modeling conducted under Task 3.1 will be refined and confirmed using detailed building prototype models. Limits for design parameter collapse indicators will be identified based on an elevated probability of collapse across a range of building prototypes.

Task 3.3: Report on Design Parameter Collapse Indicators

Results of building prototype analyses identifying design parameter collapse indicators will be summarized in a Phase 3 report suitable for publication. The purpose of this report will be to: (1) provide the engineering community with the last increment of information on the probability of collapse for vulnerable nonductile concrete building systems; (2) provide a means to assess collapse using design parameter collapse indicators; and (3) provide updated recommendations on how to implement this information within the context of ASCE/SEI 31 and ASCE/SEI 41.

8.4 Description of Tasks for Development of Initial Component Acceptance Criteria and Modeling Parameters

Due to the work of the NEES Grand Challenge project and availability of data from other recent research, a subset of the series of component documents, *Acceptance Criteria and Modeling Parameters for Concrete Components*, is recommended for development in a separate, near-term, phase of work. Phase 4 includes the development of Document 2 (columns), and Document 3 (beam-column joints). As currently envisioned, work will be based on the methodology for consistent selection of component acceptance criteria and modeling parameters described in Chapter 7. Phase 4 tasks are described below.

Task 4.1: Selection of column acceptance criteria and modeling parameters

This task will utilize the database of information collected under the NEES Grand Challenge project, as supplemented by the NEES Grand Challenge column testing program. It will synthesize available data into recommendations for changes to acceptance criteria and modeling parameters for concrete columns in ASCE/SEI 41. As the first such task to perform this work, it will also evaluate and refine the preliminary methodology described in Chapter 7 for the selection of component acceptance criteria and modeling parameters. Results from this work will be used as a basis for all future component tasks so that acceptance criteria and modeling parameters for the remaining components are selected on a consistent basis.

Review of the approach and resulting recommendations will be conducted at a workshop including a broad range of research and engineering practitioner stakeholders.

Task 4.2: Report on Acceptance Criteria and Modeling Parameters for Concrete Columns

Results and recommendations for improved ASCE/SEI 41 criteria will be summarized in the final report, *Acceptance Criteria and Modeling Parameters for Concrete Components: Columns*.

Task 4.3: Selection of beam-column joint acceptance criteria and modeling parameters

This task will utilize the database of information collected under the NEES Grand Challenge project, as supplemented by the NEES Grand Challenge beam-column joint testing program. It will synthesize available data into recommendations for changes to acceptance criteria and modeling parameters for concrete beam-column joints in ASCE/SEI 41. It will utilize the methodology described in Chapter 7, as refined for concrete columns under Task 4.1. Review of the resulting recommendations will be conducted at a workshop including a broad range of research and engineering practitioner stakeholders.

Task 4.4: Report on Acceptance Criteria and Modeling Parameters for Concrete Beam-Column Joints

Results and recommendations for improved ASCE/SEI 41 criteria will be summarized in the final report, *Acceptance Criteria and Modeling Parameters for Concrete Components: Beam-Column Joints*.

8.5 Description of Tasks for Development of Additional Component Acceptance Criteria and Modeling Parameters

Without the benefit of work conducted under the NEES Grand Challenge project, development of the remaining component documents is envisioned to require additional time and effort. Phase 5 includes the development of Document 4 (slab-column systems), Document 5 (walls), Document 6 (infill frames), Document 7 (beams), and Document 8 (rehabilitated components). Work will be based on the methodology for consistent selection of component acceptance criteria and modeling parameters described in Chapter 7. Phase 5 tasks are the same for each of the remaining documents, and described below. Work on additional components can be conducted in series or in parallel based on the availability of funding.

Task 5.1: Data collection and database development

This task involves a literature search for all available test data, and development of a database of available experimental results for the component of interest. The database of information should be similar to that collected by the NEES Grand Challenge for columns and beam-column joints, and must be detailed enough to determine fragility curves at the collapse limit state for each component.

Task 5.2: Selection of acceptance criteria and modeling parameters

This task will synthesize available data into recommendations for changes to acceptance criteria and modeling parameters for the component of interest in ASCE/SEI 41. It will utilize the methodology described in Chapter 7, as refined for concrete columns under Task 4.1. Review of the resulting recommendations will be conducted at a workshop including a broad range of research and engineering practitioner stakeholders.

Task 5.3: Report on Acceptance Criteria and Modeling Parameters for Additional Concrete Components

Results and recommendations for improved ASCE/SEI 41 criteria will be summarized in a final report, *Acceptance Criteria and Modeling Parameters for Concrete Components*, for each additional component of interest.

8.6 Recommended Schedule

The modular, phased approach to the work plan has been structured to provide greater flexibility in scheduling the various components of the program. A recommended schedule for the overall program is shown in Figure 8-2. A detailed schedule for the development of Document 1 in Phases 1, 2, and 3 is shown in Figure 8-3.

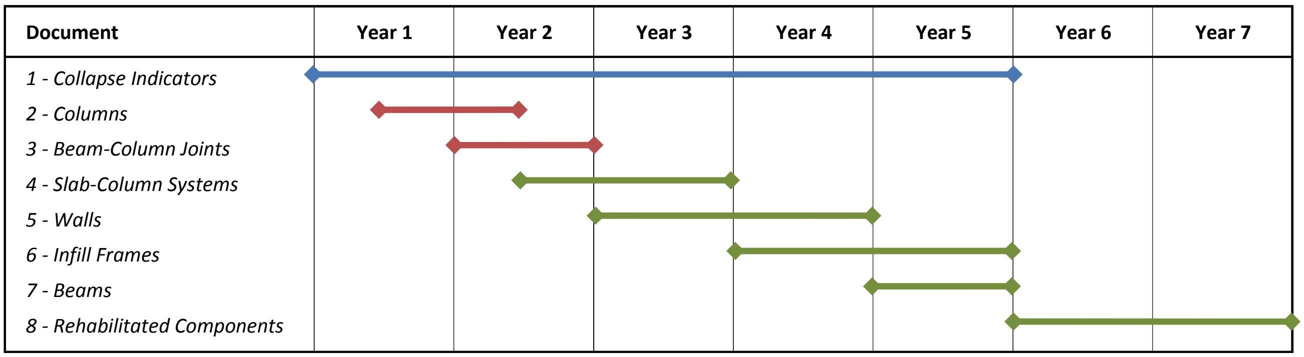

Figure 8-2 Recommended schedule of the overall program for development of Document 1 through Document 8.

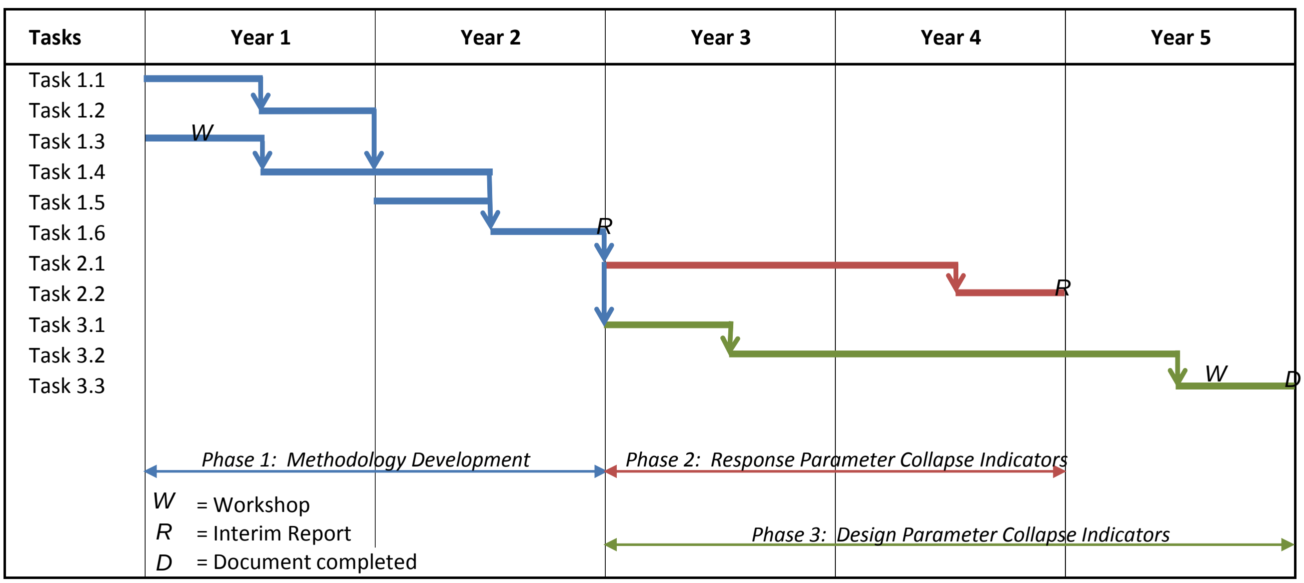

Figure 8-3 Recommended schedule for the development of Document 1 in Phases 1, 2, and 3.

With the assumption that no more than two component documents are under development at any one time, the overall program has a duration of seven years. In general, work can be conducted in parallel or in series, as funding permits. Some coordination between phases, however, is recommended.

The development of Document 1 is considered the greatest need, and is recommended as the highest priority. It has been structured to be completed in phases, with an overall duration of five years. The development of the collapse indicator methodology in Phase 1, is key to the entire program. Work to be conducted in Phase 2 and Phase 3 is directly dependent on this work.

Analytical work that will be conducted in Phase 2 and Phase 3, is related to, and would benefit from improved component acceptance criteria and modeling parameters to be developed in Phase 4 and Phase 5. Because of the importance of Document 1, however, the recommended schedule shows the development of the collapse indicator methodology concurrent with some, and preceeding most, of the component document development.

Because of the availability of information on concrete columns and beam-column joints as a result of the NEES Grand Challenge project, development of component documents under Phase 4 are recommended to occur before component documents under Phase 5. Phase 4 work on Document 2 (columns) should be staggered with work on Document 3 (beam-column joints) so that the methodology for selection of acceptance criteria and modeling parameters can be finalized.

8.7 Estimated Budget

The modular, phased approach to the work plan also provides greater flexibility in funding the various components of the program. Budget estimates for the development of Documents 1 through 8 are provided in Table 8-2. Dollar estimates for the development of Document 1, by phase, are provided in Table 8-3.

Table 8-2 Estimated Budget for Development of Document 1 through Document 8

Doc. No.	Duration (years)	Direct Technical Development	Direct Management and Oversight	Direct Expenses	Allowance for Overhead	Total cost
1	5	$1,450,000	$290,000	$406,000	$754,000	$2,900,000
2	1	$125,000	$25,000	$35,000	$65,000	$250,000
3	1	$150,000	$30,000	$42,000	$78,000	$300,000
4	1.5	$150,000	$30,000	$42,000	$78,000	$300,000
5	2	$210,000	$42,000	$58,800	$109,200	$420,000
6	2	$180,000	$36,000	$50,400	$93,600	$360,000
7	1	$125,000	$25,000	$35,000	$65,000	$250,000
8	2	$210,000	$42,000	$58,800	$109,200	$420,000
	Totals:	$2,600,000	$520,000	$728,000	$1,352,000	$5,200,000

Table 8-3 Estimated Budget for Development of Document 1 by Phase

Phase	Duration (years)	Direct Technical Development	Direct Management and Oversight	Direct Expenses	Allowance for Overhead	Total cost
1	2	$450,000	$90,000	$126,000	$234,000	$900,000
2	2	$350,000	$70,000	$98,000	$182,000	$700,000
3	3	$650,000	$130,000	$182,000	$338,000	$1,300,000
	Totals:	$1,450,000	$290,000	$406,000	$754,000	$2,900,000

Budget estimates for the development of each document have been prepared based on the estimated duration and level of effort needed to complete the tasks contributing to each phase of work. In determining the estimated level of effort, proposed work was compared to work conducted on past projects of similar scope and duration, based on the collective experience of the project team. Dollar values have been assigned based on a weighted average hourly rate for all personnel that were envisioned to participate on the developmental teams. Each table includes an allowance for management and oversight activities, expenses, and overhead charges.

The estimated budget for the overall program is $5.2 million. The estimated budget for the development of Document 1 is $2.9 million, which is the total for Phase 1 ($900,000), Phase 2 ($700,000), and Phase 3 ($1,300,000).

8.8 Key Collaborators

The problem associated with older nonductile concrete buildings has attracted the attention of a number of stakeholders who are potential collaborators on the implementation of this work plan. Successful development of the recommended guidance documents should include collaboration with these stakeholders, some of which will be providers of necessary information, or sources of supplemental funding.

National Science Foundation (NSF). Results from the NEES Grand Challenge project are essential for the execution of work under this program. The development of Document 1 will be informed and supplemented by the results of collapse simulation studies planned for the final two years of the NEES Grand Challenge. Also, the development improved acceptance criteria and modeling parameters for columns and beam-column joints will be based the work of the NEES Grand Challenge.

Additionally, development of improved acceptance criteria for other concrete components in outlying years will benefit from future component or subassembly testing funded through NSF research programs or conducted at NEES sites.

Federal Emergency Management Agency (FEMA). In 2010, FEMA initiated funding on a project to begin addressing the risks associated with older nonductile concrete buildings. The ATC-78 Project, *Identification and Mitigation of Non-ductile Concrete Buildings*, has the objective of identifying the specific characteristics of older non-ductile concrete buildings that lead to collapse. This objective, and the focused analytical work to be performed on this project, will be directly synergistic with the development of Document 1 and identification collapse indicators to be performed under Phases 1, 2, and 3 of the program.

American Concrete Institute (ACI). The ACI Committee for Seismic Repair and Rehabilitation (ACI 369) has successfully balloted a non-mandatory language version of the concrete frame provisions of ASCE/SEI 41. This is expected to be released in early 2011 by ACI under the title *ACI 369R: Guide for Seismic Rehabilitation of Concrete Frame Buildings*. This guide will provide a mechanism to disseminate research results and assist with the future update of the concrete provisions in ASCE/SEI 41. The ongoing efforts of the committee related to updating the provisions for columns and beam-column joints are directly synergistic with the development of improved acceptance criteria and modeling provisions in Document 2 (columns) and Document 3 (beam-column joints).

EERI Concrete Coalition. The Concrete Coalition has been formative in the movement to address the risks associated with older nonductile concrete buildings, and should be involved with the development of recommended documents. The Concrete Coalition can facilitate broad input from the research and engineering community, which is key to the successful development of nationally accepted guidance documents, and can mobilize engineering practitioners, researchers, and students in the collection of necessary information.

8.9 Implementation in Codes and Standards

Implementation of the resulting guidance documents into codes and standards can take many forms, including: (1) translation of guidelines into prestandards for use by model codes and standards development agencies; (2) advocacy in the code change process for direct adoption into model building codes; or (3) collaboration with standards development committees for direct adoption into national consensus standards.

It is possible that some content in the resulting guidance documents will not be suitable for standardization. Existing buildings frequently result in situations that defy standard practices, and require the flexibility of guidelines, combined with engineering judgment derived from experience, to arrive at optimal seismic risk mitigation solutions.

Selected tasks in this program will result in recommendations for improving current seismic evaluation and rehabilitation standards. While new technical information will be developed, adoption of information into these standards is envisioned as a separate step from the development of the guidance documents described herein, and has not been estimated as part of the current program budget. It is recommended that the best approach for implementation into codes and standards be decided once the guidance documents have been completed.

Appendix A

Draft Outline - Assessment of Collapse Potential and Mitigation Strategies

This appendix provides a draft outline for *Assessment of Collapse Potential and Mitigation Strategies*, which is the first in a series of documents to be developed under the umbrella title *Guidance for Collapse Assessment for Existing Reinforced Concrete Buildings*.

1. Introduction

1.1 Background

1.2 Objectives and Scope of the document

- Goal is to establish collapse indicators that are suitable for rapid assessment and for detailed assessment.
- Identify parameters (collapse indicators) that are correlated with elevated collapse probability
- Propose limits on collapse indicators for assessment
- Include collapse mitigation strategies

1.3 Use of this document to mitigate collapse risk

2. Observations from Past Earthquakes

2.1 Examples of building collapses in past earthquakes

2.2 Component and system characteristics that have been shown to trigger collapse

3. Methodology to Identify Limits for Collapse Indicators

3.1 Methodology for determining probability of collapse

3.2 Simplified analyses

- Simplified models representing a class of buildings where collapse indicator parameters can be easily changed.

3.3 Building prototype models

- Library of detailed models for real reinforced concrete buildings capable of capturing collapse
- Variation in design parameters to investigate influence of collapse indicators and identify limits

3.4 Summary of results

4. Assessment Procedure – Recommended Limits on Collapse Indicators or Combinations of Collapse Indicators

4.1 Design Parameter Collapse Indicators

- Rapid assessment
- Engineering calculations

4.2 Response Parameter Collapse Indicators based on detailed analyses

5. Common Deficiencies in Nonductile Concrete Buildings and Cost-Effective Mitigation Strategies

5.1 Shear-Critical Columns
5.2 Unconfined Beam-Column Joints
5.3 Slab-Column Connections
5.4 Splice and Connectivity Weakness
5.5 Weak-Story Mechanism
5.6 Overall Weak Frames
5.7 Overturning Mechanisms
5.8 Severe Plan Irregularity
5.9 Severe Vertical Irregularity
5.10 Pounding

6. Conclusions

Appendix A. Results from studies using simplified models

Appendix B. Results from studies using building prototypes

Appendix C. Case Studies

Appendix B

Draft Outline - Acceptance Criteria and Modeling Parameters for Concrete Components: Columns

This appendix provides a draft outline for *Acceptance Criteria and Modeling Parameters for Concrete Components: Columns*, which is the second in a series of documents to be developed under the umbrella title *Guidance for Collapse Assessment for Existing Reinforced Concrete Buildings*.

1. Introduction

1.1 Objectives and Scope

- Synthesize research results from NEES Grand Challenge testing on collapse behavior of concrete columns
- Recommend changes to acceptance criteria and modeling parameters in ACI 369R and ASCE/SEI 41
- Focus on collapse prevention acceptance criteria and practical modeling recommendations
- Assess impact of column splices

1.2 Observations from past earthquakes

- Summary of building collapses precipitated by column failures

2. Literature Review

2.1 Laboratory tests on columns

- Summary of past tests including shear and axial load failure

2.2 Capacity models for shear and axial load failure

- Focus on deformation capacity models

2.3 Acceptance criteria in available standards and guidelines

- ASCE/SEI 41
- ACI 369R methodology for selecting modeling parameters and acceptance criteria

2.4 Modeling techniques for columns (flexural – shear – axial models)

- Models used in practice
- Models used in research

3. Database of Column Tests for Shear and Axial Load Failure

3.1 General observations and trends from available data

3.2 Comparisons with ASCE/SEI 41

4. Recommended Modifications to ACI 369R and ASCE/SEI 41

4.1 Distributions for “a” and “b” values (median and coefficient of variation)
4.2 Consideration of bidirectional demands and capacities
4.3 Recommendations for appropriate degree of conservatism on acceptance criteria for different failure modes
4.4 Modeling recommendations for columns experiencing shear and axial failure

5. Conclusions

Appendix A. Laboratory Tests from NEES-Grand Challenge Project

A.1 Objectives
A.2 Description of tests
A.3 Summary of results

Appendix B. Other Laboratory Tests

B.1 Objectives
B.2 Description of tests
B.3 Summary of results

Appendix C

Draft Outline - Acceptance Criteria and Modeling Parameters for Concrete Components: Beam-Column Joints

This appendix provides a draft outline for *Acceptance Criteria and Modeling Parameters for Concrete Components: Beam-Column Joints*, which is the third in a series of documents to be developed under the umbrella title *Guidance for Collapse Assessment for Existing Reinforced Concrete Buildings*.

1. Introduction

1.1 Objectives and Scope of the document
- Synthesize research results from NEES Grand Challenge on collapse behavior of concrete beam-column joints
- Recommend changes to acceptance criteria and modeling parameters in ACI 369R and ASCE/SEI 41
- Focus on collapse prevention acceptance criteria and practical modeling
- Consider both shear and axial failure

1.2 Observations from past earthquakes
- Summary of building collapses precipitated by joint failures

2. Literature review

2.1 Laboratory tests on interior beam-column joints (confined on all four sides)

2.2 Laboratory tests on exterior beam-column joints

2.3 Laboratory tests on corner beam-column joints

2.4 System tests with joint failures
- Influence of joint failures on frame response

2.5 Capacity models for shear and axial load failure
- Shear strength capacity models
- Deformation capacity models

2.6 Acceptance criteria in available standards and guidelines
- ASCE/SEI 41
- ACI 369R methodology for selecting modeling parameters and acceptance criteria

2.7 Modeling techniques for beam-column joints
- Models used in practice
- Models used in research

3. Database of beam-column joint tests

3.1 General observations/trends from database

3.2 Comparisons with ASCE/SEI 41

4. Recommended modifications to ACI 369R and ASCE/SEI 41

4.1 Changes to shear strength model

4.2 Distributions for “a” and “b” values (median and coefficient of variation)

4.3 Recommendations for appropriate degree of conservatism on acceptance criteria for different failure modes

4.4 Modeling recommendations for joints experiencing shear and axial failure - including models linking behavior of beams/joints and columns/joints

5. Conclusions

Appendix A. Laboratory tests from NEES-GC

A.1 Objectives

A.2 Description of tests

A.3 Summary of results

Appendix B. Other Laboratory Tests

B.1 Objectives

B.2 Description of tests

B.3 Summary of results

References

ASCE, 2003, *Seismic Evaluation of Existing Buildings*, ASCE/SEI 31-03, American Society of Civil Engineers/Structural Engineering Institute, Reston, Virginia.

ASCE, 2006, *Minimum Design Loads for Buildings and Other Structures*, ASCE Standard ASCE/SEI 7-05, including Supplement No. 1, American Society of Civil Engineers/Structural Engineering Institute, Reston, Virginia.

ASCE, 2007a, *Seismic Rehabilitation of Existing Buildings*, ASCE Standard ASCE/SEI 41-06, American Society of Civil Engineers/Structural Engineering Institute, Reston, Virginia.

ASCE, 2007b, *Supplement No. 1 to ASCE 41*, available at http://content.seinstitute.org/publications/ASCE41supplement.html. (Not included in the current printing of ASCE/SEI 41-06).

ATC, 1987, *Evaluating the Seismic Resistance of Existing Buildings*, ATC-14 Report, Applied Technology Council, Redwood City, California.

ATC, 2003, *The Missing Piece: Improving Seismic Design and Construction Practices*, ATC-57 Report, Applied Technology Council, Redwood City, California.

ATC, 2009a, *Guidelines for Seismic Performance Assessment of Buildings*, ATC-58 Report, 50% Draft, prepared by the Applied Technology Council for the Federal Emergency Management Agency, Redwood City, California.

ATC, 2009b, *NEHRP Workshop on Meeting the Challenges of Existing Buildings, Part 2:Status Report on Seismic Evaluation and Rehabilitation of Existing Buildings*, ATC-71 Report, prepared by the Applied Technology Council for the Federal Emergency Management Agency, Redwood City, California.

Shoraka, B.M., Elwood, K.J., Haukaas T., 2010, “System-level Acceptance Criteria for Seismic Assessments of Pre-1970s Reinforced Concrete Buildings,” *9th US National and 10th Canadian Conference on Earthquake Engineering*, July 25-29, Toronto, Canada.

Brena, S. and Alcocer, S., 2009, “Seismic Performance Evaluation of Rehabilitated Reinforced Concrete Columns through Jacketing,” *Proceedings of the ATC-SEI Conference on Improving the Seismic Performance of Existing Buildings and Other Structures*, December 9-11, 2009, San Francisco, California.

CSSC, 1995, *Northridge Earthquake – Turning Loss to Gain*, Report No. 95-01, California Seismic Safety Commission, Sacramento, California.

CEN, 2005, *Eurocode 8: Design of Structures for Earthquake Resistance – Part 3: Assessment and Retrofitting of Buildings*, EN 1998-3, European Committee for Standardization, Brussels, Belgium.

Chassioti, S.G., Syntzirma, D.V., and Pantazopoulou, S.J., 2010, "Codes of Assessment for Buildings: A Comparative Study," *9th US National Conference on Earthquake Engineering*, Toronto, Canada.

Concrete Coalition, 2010, http://www.concretecoalition.org/.

DOD, 2005, *Seismic Review Procedures for Existing Military Buildings*, Unified Facilities Criteria (UFC) 3-330-03A, Washington, D.C.

DOD, 2006, *Structural Engineering*, Unified Facilities Criteria (UFC) 3-300-10N, Washington, D.C.

DOD, 2007, *Seismic Design for Buildings*, Unified Facilities Criteria (UFC) 3-310-04, Washington, D.C.

Elwood, K.J., 2004, "Modeling failures in existing reinforced concrete columns". *Canadian Journal of Civil Engineering*; 31(5): pp.846–859.

Elwood, K.J., and Moehle, J.P., 2005, "Axial Capacity Model for Shear-Damaged Columns," *Structural Journal*, ACI, V. 102, No. 4, pp. 578-587.

Elwood, K.J., and Moehle, J.P., 2008, "Dynamic Collapse Analysis for a Reinforced Concrete Frame Sustaining Shear and Axial Failure," *Earthquake Engineering and Structural Dynamics*, vol. 37, no. 7, pp. 991-1012.

Elwood, K.J., Matamoros, A., Wallace, J.W., Lehman, D.E., Heintz, J.A., Mitchell, A., Moore, M.A., Valley, M.T., Lowes, L. Comartin, C., and Moehle, J.P., 2007, "Update of ASCE/SEI 41 Concrete Provisions", *Earthquake Spectra*, Earthquake Engineering Research Institute, August 2007, vol. 23, no. 3, pp. 493-523.

FEMA, 1992, *NEHRP Handbook for Seismic Evaluation of Existing Buildings*, FEMA 178, Federal Emergency Management Agency, Washington, D.C.

FEMA, 1997, *NEHRP Guidelines for the Seismic Rehabilitation of Buildings*, FEMA 273, Federal Emergency Management Agency, Washington, D.C.

FEMA, 1998, *Handbook for the Seismic Evaluation of Buildings - A Prestandard*, FEMA 310, Federal Emergency Management Agency, Washington, D.C.

FEMA, 2000, *Prestandard and Commentary for the Seismic Rehabilitation Of Buildings*, FEMA 356, Federal Emergency Management Agency, Washington, D.C.

FEMA, 2006, *Techniques for Seismic Rehabilitation of Existing Buildings*, FEMA 547, Federal Emergency Management Agency, Washington, D.C.

FEMA, 2009a, *Engineering Guideline for Incremental Seismic Rehabilitation*, FEMA P-420, Federal Emergency Management Agency, Washington, D.C.

FEMA, 2009b, *Effects of Strength and Stiffness Degradation on Seismic Response*, FEMA P-440A, Federal Emergency Management Agency, Washington, D.C.

Gulec, C. K., Whittaker, A. S., and Stojadinovic, B., 2008, "Shear Strength of Squat Rectangular Reinforced Concrete Walls," *ACI Structural Journal*, V. 105, No. 4, July-Aug. 2008, pp. 488-497.

Gillengerten, J., 2006, Personal communication, District Structural Engineer, Office of Statewide Health Planning and Development (OSHPD), Sacramento, California.

ICC, 2009a, *International Building Code*, International Code Council, Washington, D.C.

ICC, 2009b, *International Existing Building Code*, International Code Council, Washington, D.C.

JBDPA, 2001, *Standard for Evaluation of Existing Reinforced Concrete Buildings*, Japan Building Disaster Prevention Association, Tokyo, Japan.

Kang T. H.-K., and Wallace J. W., 2006, "Punching of Reinforced and Post-Tensioned Concrete Slab-Column Connections," *ACI Structural Journal*, American Concrete Institute, Detroit, Michigan, V. 103, No. 4, pp. 531-540.

Kang T. H.-K., Wallace J. W. and Elwood, K. J., 2009, "Nonlinear Modeling of Flat Plate Systems," *Journal of Structural Engineering*, ASCE, vol. 135, no. 2.

Mazzoni, S., et al., 2006, *User Command-Language Manual* (OpenSees Version 1.7.3) Open System for Earthquake Engineering Simulation, Pacific Earthquake Engineering Research Center, University of California, Berkeley, http://opensees.berkeley.edu/OpenSees/manuals/usermanual/index.html.

Moehle, J., 2007, *The Top Ten List*, presentation at the Concrete Coalition Meeting in November, 2007, Los Angeles, California.

NIST, 2002, *Standards of Seismic Safety for Existing Federally Owned and Leased Buildings*, ICSSC RP 6/NISTIR 6762, National Institute of Standards and Technology, Gaithersburg, Maryland.

NZSEE, 2006, *Assessment and Improvement of the Structural Performance of Buildings in Earthquakes*, New Zealand Society for Earthquake Engineering, Upper Hutt, New Zealand.

Otani S., 1999, “RC building damage statistics and SDF response with design seismic forces,” *Earthquake Spectra*, 15(3): pp.485–501.

OSHPD, 2009, *Seismic Retrofit Program Overview*, California Office of Statewide Heath Planning and Development, http://www.oshpd.cahwnet.gov/FDD/SB1953/.

Panagiotakos, T. B., and Fardis, M. N.,2001, “Deformation of Reinforced Concrete Members at Yielding and Ultimate,” *ACI Structural Journal*, American Concrete Institute, Detroit, Michigan, V. 98, No. 2, pp. 135-148.

Sezen, H., Dragovich, J., Ghannoum, W., Lowes, L.N., Brena, S.F., Elwood, K.J. “Guide for Seismic Rehabilitation of Concrete Buildings: Future Changes,” *Proceedings of the ATC-SEI Conference on Improving the Seismic Performance of Existing Buildings and Other Structures*, December 9-11, 2009, San Francisco, California.

Shing, P.B., Stavridis, A., Koutromanos, I., Willam, K., Blackard, B., Kyriakides, M. Billington, S., and Arnold, S., 2009, “Seismic Performance of Non-Ductile RC Frames with Brick Infill,” *Proceedings of the ATC-SEI Conference on Improving the Seismic Performance of Existing Buildings and Other Structures*, December 9-11, 2009, San Francisco, California.

Talaat, M., and Mosalam, K.M., 2009, “Modeling Progressive Collapse in Reinforced Concrete Buildings using Direct Element Removal,” *Earthquake Engineering Structural Dynamics*, 38(5), pp.609-634.

Vamvatsikos, D. and Cornell, C.A., 2006, “Direct estimation of the seismic demand and capacity of oscillators with multi-linear static pushovers through Incremental Dynamic Analysis,” *Earthquake Engineering and Structural Dynamics*, 35(9): 1097-1117.

Yavari, S., Elwood, K.J., and Wu, C.L., 2009, “Collapse of a Nonductile Concrete Frame: Evaluation of Analytical models,” *Earthquake Engineering and Structural Dynamics*, vol. 38, no. 2, pp. 225-241.

Project Participants

National Institute of Standards and Technology

John (Jack) R. Hayes
Building and Fire Research Laboratory (MS8604)
National Institute of Standards and Technology
100 Bureau Drive
Gaithersburg, Maryland 20899

Jeffrey J. Dragovich
Building Fire and Research Laboratory (MS8603)
National Institute of Standards and Technology
100 Bureau Drive
Gaithersburg, Maryland 20899

NEHRP Consultants Joint Venture

APPLIED TECHNOLOGY COUNCIL
201 Redwood Shores Parkway, Suite 240
Redwood City, California 94065
www.ATCouncil.org

CONSORTIUM OF UNIVERSITIES FOR
RESEARCH IN EARTHQUAKE ENGINEERING
1301 S. 46th Street, Building 420
Richmond, California 94804
www.CUREE.org

Joint Venture Management Committee

James R. Harris
J.R. Harris & Company
1776 Lincoln Street, Suite 1100
Denver, Colorado 80203

Christopher Rojahn
Applied Technology Council
201 Redwood Shores Parkway, Suite 240
Redwood City, California, 94065

Robert Reitherman
Consortium of Universities for Research in
Earthquake Engineering
1301 S. 46th Street, Building 420
Richmond, California 94804

Andrew Whittaker
University at Buffalo
Dept. of Civil, Structural, and Environ. Engin.
230 Ketter Hall
Buffalo, New York 14260

Joint Venture Program Committee

Jon A. Heintz (Program Manager)
Applied Technology Council
201 Redwood Shores Parkway, Suite 240
Redwood City, California 94065

William T. Holmes
Rutherford and Chekene
55 Second Street, Suite 600
San Francisco, California 94105

C.B. Crouse
URS Corporation
1501 4th Ave. Suite 1400
Seattle, Washington 98101

Jack Moehle
University of California, Berkeley
325 Davis Hall – MC 1792
Berkeley, California 94720

Michael Constantinou
University at Buffalo
Dept. of Civil, Structural and Environ. Engin.
132 Ketter Hall
Buffalo, New York 14260

James R. Harris (ex-officio)
Andrew Whittaker (ex-officio)

Project Technical Committee

Ken Elwood (Technical Director)
University of British Columbia
6250 Applied Science Lane
Vancouver, British Columbia V6T 1Z4

Craig Comartin
CDComartin Inc.
7683 Andrea Avenue
Stockton, California 95207

William T. Holmes
Rutherford and Chekene
55 Second Street, Suite 600
San Francisco, California 94105

Dominic Kelly
Simpson Gumpertz & Heger
41 Seyon Street, Building 1, Suite 500
Waltham, Massachusetts 02453

Laura Lowes
University of Washington
233C More Hall, Box 352700
Seattle, Washington 98195

Jack Moehle
University of California, Berkeley
325 Davis Hall – MC 1792
Berkeley, California 94720

Project Review Panel

Nathan Gould
ABS Consulting
77 Wesport Plaza, Suite 210
St. Louis, Missouri 63146

Afshar Jalalian
Rutherford and Chekene
55 Second Street, Suite 600
San Francisco, California 94105

Jim Jirsa
University of Texas
Department of Civil Engineering, ECJ 4.2
Austin, Texas 78712

Terry Lundeen
Coughlin Porter Lundeen
413 Pine Street, Suite 300
Seattle, Washington 98101

Mike Mehrain
URS Corporation
915 Wilshire Blvd., Suite 700
Los Angeles, California 90017

Julio Ramirez
Purdue University
Neil Armstrong Hall of Engineering, Suite 2000
701 West Stadium Ave.
West Lafayette, Indiana 47907

Project Manager

David A. Hutchinson
Buehler & Buehler, Structural Engineers
600 Q Street, Suite 200
Sacramento, California 95811