Guidelines for Post-Earthquake Repair and Retrofit of Buildings Based on Assessment of Performance-Critical Damage

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Cover photograph – Older concrete building after the 2010 Maule Earthquake, previously damaged and repaired following the 1985 Chile Earthquake, located in Vina del Mar, Chile
(photo credit: J. Heintz).

Foreword

Repair of damaged buildings is critical for community recovery after earthquake disasters, and in turn, for overall resilience. As part of its responsibilities under the National Earthquake Hazards Reduction Program (NEHRP), and in accordance with the National Earthquake Hazards Reduction Act of 1977 (PL 94-125, as amended), the Federal Emergency Management Agency (FEMA) originally initiated a project on assessment and repair of earthquake damaged concrete and masonry wall buildings after the 1994 Northridge Earthquake. This project resulted in three documents that were published in 1998: (1) FEMA 306, *Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings, Basic Procedures Manual*; (2) FEMA 307, *Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings, Technical Resources*; and (3) FEMA 308, *The Repair of Earthquake Damaged Concrete and Masonry Wall Buildings*. This series of documents provided a groundbreaking framework for detailed engineering assessments of residual strength in earthquake-damaged concrete wall buildings and set the state of practice for seismic structural engineering in the United States for more than 20 years.

Findings and research completed in New Zealand after the 2011 Christchurch Earthquake, as well as new information and an evolving understanding of the effects of damage to structural components that have experienced strong shaking and their residual capacities for future earthquakes, prompted a review and update of the FEMA documents. This work, which started in 2018, was conducted by highly respected members of the structural engineering communities in the United States and New Zealand, some of whom were also involved in the original project that produced FEMA 306/307/308. This report presents the resulting next-generation guidance with criteria for assessing and repairing earthquake-damaged buildings, including a new concept and technical term, *Performance-Critical Damage*. This publication partly replaces the FEMA 306/307/308 series addressing concrete wall buildings and expands the scope to concrete frames. Unlike FEMA 306/307/308, this new publication does not include masonry buildings. It is FEMA NEHRP's intent to publish subsequent editions addressing masonry and steel buildings.

FEMA acknowledges the Applied Technology Council, the Project Technical Committee, Project Review Panel, and Working Group members. FEMA also gratefully recognizes the Natural Hazards Commission Toka Tū Ake of New Zealand, which provided funding for the involvement of Dr. Nicholas Brooke, a New Zealand engineering practitioner with first-hand experience of the assessment and repair challenges following the 2011 Christchurch Earthquake. Much of the new ground was explored and developed across many time zones. The project was incredibly lucky to have internationally recognized participants. Additionally, FEMA would like to thank the Trial Users, who provided thoughtful feedback and reinforcing excitement for the new methodology. All who participated in this project provided amazing creativity and patience. They are listed at the end of this report.

FEMA also recognizes Michael Mahoney, who retired from FEMA during this project, for setting this project up for success and his incredible mentoring, as well as William T. Holmes for acting as FEMA NEHRP's Subject Matter Expert for this project. His quiet steadiness, dedication, and experience-based wisdom made a significant impact on the project.

Federal Emergency Management Agency

Preface

In 2018, the Federal Emergency Management Agency awarded the Applied Technology Council the first in a series of task orders under contract HSFE60-17-D-0002 to develop state-of-the-art guidance for post-earthquake repair and retrofit decision-making. FEMA P-2335 is the result of this work.

The report establishes a new methodology and procedures that will bring more consistency and reliability to repair and retrofit decisions made by structural engineers following damaging earthquakes. An essential concept in the methodology is Performance-Critical Damage, which is damage that leads to a reduction in component and building strength, deformation capacity, or both and, as a result, the future seismic performance of the building is impaired.

FEMA P-2335 addresses reinforced concrete frame and wall construction, but information on other structural materials and building systems may be added in future editions.

ATC is indebted to the leadership of Ken Elwood and Abbie Liel, Co-Project Technical Directors, and to the other members of the Project Technical Committee, including Nicholas Brooke, Greg Deierlein, Jack Moehle, Bill Tremayne, and John Wallace, who managed and performed the technical development effort. The participation of Nicholas Brooke was made possible with funding from the Natural Hazards Commission Toka Tū Ake of New Zealand. The Project Review Panel, consisting of Jim Malley and Santiago Pujol, provided technical review and advice at key stages of the work.

FEMA P-2335 was developed over six years and included contributions from 17 Project Working Group members. The development effort included testing of the methodology and procedures, review, and comment in a workshop setting. Eleven engineers participated as Trial Users, providing valuable feedback that led to important enhancements.

ATC gratefully acknowledges Christina Aronson (FEMA Project Officer) and Bill Holmes (Subject Matter Expert) for their leadership, input, and guidance in the preparation of this report. Chiara McKenney managed the project during critical phases of its development. The names and affiliations of all who contributed to this report are provided in the list of Project Participants at the end of this report.

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ATC Director of Projects

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Executive Summary

When the next large, damaging earthquake occurs in the United States, engineers, building owners. and communities will face substantial uncertainty about what buildings need to be repaired, retrofitted, or demolished. This uncertainty could result in buildings not being repaired that should be, long delays in needed repairs and retrofits, and unnecessary demolition. Reducing this uncertainty is essential for community resilience and recovery.

FEMA P-2335, *Guidelines for Post-Earthquake Repair and Retrofit of Buildings Based on Assessment of Performance-Critical Damage,* provides guidelines that can be used after an earthquake to identify buildings for which repair or repair and retrofit is critical. It also summarizes information about repair strategies that can be used to achieve the necessary performance objectives for repair. In doing so, the *Guidelines* update existing earthquake repair guidance provided by FEMA for reinforced concrete wall buildings, as well as extend the scope to include concrete frame systems, and ensure that this guidance is consistent with modern building practices and understanding of seismic performance. The *Guidelines* do not establish policy related to post-earthquake assessment, repair, or retrofit of buildings. Rather, they provide technical criteria that can be used to implement such policies, such as those articulated in the *International Existing Building Code* (IEBC), which is the governing reference standard in many U.S. jurisdictions. The intended users of FEMA P-2335 are engineers with experience in seismic design and assessment and building officials with authority over such projects.

The purpose of FEMA P-2335 is to provide a procedure that can be used to determine the need for post-earthquake repair or repair and retrofit. These procedures are intended to inform the identification of damage, the evaluation of observed damage caused by earthquakes in terms of their effects on building seismic performance, the determination of the need for repair or repair and retrofit, and the identification of repair measures.

The goal of the procedures in FEMA P-2335 is to identify damage that leads to a reduction in component and/or building strength and/or deformation capacity. This damage is referred to as *Performance-Critical Damage*. As a result of such damage, the future seismic performance of the building is impaired, leading to elevated collapse risk and amplified drift demands relative to the pre-earthquake condition. This damage therefore indicates the need for performance-critical repairs. The definition of Performance-Critical Damage, which corresponds to component damage that meets or exceeds the onset of strength loss, is the result of extensive background work, including the review of experimental results and the completion of a suite of analytical studies.

The *Guidelines* are organized with five chapters, four appendices, and extensive electronic resources. Chapter 1 through Chapter 4 outline the principles of post-earthquake repair decision-making, which apply regardless of building material and system. Chapter 5 provides extensive qualitative and quantitative information defining criteria needed to apply these principles to reinforced concrete buildings (wall and frame).

In the *Guidelines*, Performance-Critical Damage to buildings is identified through inspection and analysis (Chapter 3). The goal of this step is to identify all possible locations of potential Performance-Critical Damage and to document this damage with sufficient detail such that this damage can be subsequently evaluated. This identification emphasizes visual inspection. Three different levels of inspection are described, which involve varying levels of disruption. For buildings where detailed inspection of the structural components is impeded by architectural enclosures or other obstructions, a structural analysis of the building under the damaging earthquake shaking may be performed to help identify damage.

Once damage is identified, FEMA P-2335 provides evaluation procedures (Chapter 4) to determine whether the earthquake-damaged building needs repair to restore strength and deformation capacity (i.e., performance-critical repair). This process involves, first, classification of damage to earthquake-damaged components identified through inspection. This damage classification is made through reference to descriptions and images of Performance-Critical Damage for each component type provided in FEMA P-2335; these materials are provided in Chapter 5 and in electronic resources for reinforced concrete components and systems.

Component damage classifications provide the basis for the determination of the Building Repair Outcome. Performance-critical repair is needed if any component has Performance-Critical Damage. A building may also be found to need both repair and retrofit. In the IEBC, a determination if retrofit is needed is made through evaluation of compliance with acceptable codes and standards, and identification of substantial structural damage and disproportionate earthquake damage. FEMA P-2335 provides guidance on the use of damage classifications to inform the determination as to whether substantial structural damage or disproportionate earthquake damage has occurred.

If Performance-Critical Damage is found and repair is therefore warranted, FEMA P-2335 also provides guidance on repair techniques. For reinforced concrete components, the *Guidelines* document whether the repair techniques for concrete and reinforcement act as a repair to restore appearance or durability, a performance-critical repair, or both. FEMA P-2335 also categorizes repair types by damage type and provides descriptions of the major repair techniques.

These *Guidelines* are applicable to buildings in any seismic design category. The procedures primarily focus on damage to building structures due to ground shaking, which is a critical source of damage in an earthquake.

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Chapter 1: Introduction

1.1 Introduction

The purpose of these *Guidelines* is to guide the assessment of buildings to determine the need for post-earthquake repair or repair and retrofit. The procedures in these *Guidelines* are intended to inform the identification of damage, the evaluation of observed damage caused by earthquakes in terms of their effects on building seismic performance, the determination of the need for repair or repair and retrofit, and the identification of repair measures.

The intended users of this document are engineers with experience in seismic design and assessment and building officials with authority over such projects. Information in these *Guidelines* might also be useful to building owners, insurance adjusters, and government agencies. Such users should consult with a qualified engineer for application or interpretation of the information contained herein.

1.2 Scope

These *Guidelines* address building structures damaged by earthquake ground shaking. They provide a framework for post-earthquake assessment applicable to buildings in general. Detailed criteria in these *Guidelines* are limited to reinforced concrete frame and wall construction, including modern and older existing buildings. Information on other structural materials and building systems may be added in future editions.

These *Guidelines* are applicable to buildings in any seismic design category. They focus on damage to building structures due to ground shaking, which is a critical source of damage in an earthquake. Other earthquake effects, such as surface fault rupture, foundation settlement, liquefaction, lateral spreading, and landslide, are not directly considered. However, the procedures herein can be used to identify, evaluate, and determine necessary repair actions for damage to the superstructure from these effects occurring alone, or in combination with, ground shaking. Fire-following earthquake is not considered.

Earthquakes can cause damage to structural and nonstructural components, as well as contents, of buildings. These *Guidelines* address damage to structural components, and do not provide direct guidance for assessment, repair, or retrofit of secondary structures, nonstructural components, or contents. However, the identification and evaluation procedures for structural components provided herein can provide useful context for the assessment of damage to other components and systems.

These *Guidelines* focus on the identification and mitigation of *Performance-Critical Damage*. Performance-Critical Damage is a concept defined herein that is damage that leads to a reduction in component and building strength, deformation capacity, or both. As a result of such damage, the future seismic performance of the building is impaired, implying elevated collapse risk and amplified drift demands, relative to the pre-earthquake condition, and indicating the need for performance-critical repairs. These *Guidelines* also identify repair strategies that can address cosmetic, durability, or serviceability concerns from the damaging earthquake.

These *Guidelines* cover post-earthquake *assessment*, *repair*, and *retrofit*, where:

- Post-earthquake *assessment* is the process of determining whether an earthquake has negatively affected the performance of a building in terms of drift demands and collapse risk in future earthquakes. It includes: identification of earthquake damage; structural analysis used to guide inspection or damage assessment; classification of component damage; and determination of necessary mitigation actions. Typically, assessment would occur after a rapid post-earthquake safety evaluation, such the procedures found in ATC-20-1, *Field Manual: Postearthquake Safety Evaluation of Buildings, 2nd Edition* (ATC, 2005), but it does not depend on the outcome of such an evaluation.

- Post-earthquake *repair* corrects damage to a structural component, element, or system without substantial increases in stiffness, strength, or deformation capacity, or changes to the load path. *Performance-critical repairs* are repairs that restore component and building strength and deformation capacity to return the pre-earthquake performance of a building in terms of drift control and collapse risk. Other repairs might also be warranted to restore appearance, durability, or serviceability conditions.

- Post-earthquake *retrofit* increases stiffness, strength, deformation capacity, energy dissipation capacity, or a combination of these, for a system, or changes its load path relative to the pre-earthquake condition. A retrofit can be carried out along with repair. These *Guidelines* include procedures for determining the necessity of seismic retrofit of an earthquake-damaged building, but they do not include procedures for designing the seismic retrofit. These *Guidelines* refer to ASCE/SEI 41, *Seismic Evaluation and Retrofit of Existing Buildings* (ASCE, 2023), for retrofit provisions that can be applied to buildings evaluated in accordance with these *Guidelines*.

1.3 Organization and Content

These *Guidelines* outline procedures for post-earthquake identification of damage, performance assessment, determination of the need for repair or repair and retrofit, and the identification of repair measures for earthquake-damaged buildings.

- Chapter 2, *Overview,* provides an overview of the process for post-earthquake assessment, repair and retrofit, showing how damage identification is used to determine damage classifications for building components that are used to evaluate repair and retrofit needs.

- Chapter 3, *Identification of Earthquake Damage,* describes the process of identifying earthquake damage through inspection and, in some cases, structural analysis.

- Chapter 4, *Evaluation of Earthquake Damage*, describes the classification of earthquake damage to components and the determination of the need for repair or repair and retrofit.

- Chapter 5, *Reinforced Concrete*, contains the necessary material-specific criteria for assessment of reinforced concrete frame and wall construction, including failure mode identification, component damage limits, and guidance on the use of visual damage databases to classify damage. This chapter also describes repair techniques for these components and their suitability for addressing Performance-Critical Damage and other repair objectives. The criteria in Chapter 5 can be used for reinforced concrete frame and wall components even if some elements in the building, such as a wood diaphragm, are not reinforced concrete.

Each of these chapters is written with guidelines text and commentary. The goal of this organization is to provide streamlined guidelines and steps for the engineer, while providing commentary that provides more details regarding the steps to be taken and their rationale.

These *Guidelines* are accompanied by extensive electronic resources, including visual damage state databases and summaries of laboratory tests of repaired reinforced concrete components.

The appendices to these *Guidelines* provide additional information including background on the development of various components of the procedures. A glossary, list of symbols used throughout these *Guidelines*, references, and list of project participants are provided at the end.

1.4 Policy Considerations

The post-earthquake assessment, repair, and retrofit process outlined in these *Guidelines* is prompted by the occurrence of a damaging earthquake followed by a request or requirement for damage evaluation. The request or requirement for damage evaluation can be made by an individual owner or by the Authority Having Jurisdiction. It is not the intent of these *Guidelines* to establish policy related to post-earthquake evaluation, repair, or retrofit of buildings by specifying under what conditions these requests or requirements should be made. Rather, they intend to provide guidance to the responsible engineer in implementing post-earthquake assessment, repair, and retrofit for a building that is identified for damage evaluation. The evaluating engineer should review all regulations and requirements from the Authority Having Jurisdiction in performing a requested or required damage evaluation.

Post-earthquake damage assessment, repair, and retrofit can occur as a voluntary mitigation action or a triggered mitigation action. *Voluntary mitigation* is undertaken at the discretion of a building owner or other stakeholder and may be driven by concerns about deficiencies in the structure that existed before the earthquake or about continued building function after the earthquake. A key point is that the mitigation is voluntary with respect to the Authority Having Jurisdiction, such that there may be considerable leeway in determining the mitigation performance objectives. *Triggered mitigation* is required by a standing regulation when certain pre-defined conditions, such as a damaging earthquake, occur. Because triggered mitigation involves compliance with the standing regulation or building code, the regulation or code mandates the triggering conditions, the scope of the triggered work, and the criteria for that work. In that case, the minimum performance objectives may be more narrowly and precisely defined. Both voluntary and triggered mitigation actions may then require other work as determined by the Authority Having Jurisdiction; these actions are not addressed here. These *Guidelines* do not address decisions related to demolition in lieu of repair or retrofit. In some cases, demolition may be the most prudent choice when considering the costs of repair or retrofit and the expected future performance after repair or retrofit.

The applicable code for repair and retrofit in many places in the United States is an adopted version of the *International Existing Building Code* (IEBC) (ICC, 2024a). These *Guidelines* are consistent with the concepts outlined in the IEBC provisions, including substantial structural damage and disproportionate earthquake damage, and provide guidance on how to use the damage identification and assessments described herein to make these determinations. Interpretations of state and local ordinances that modify model code upgrade requirements are outside the scope of these *Guidelines*.

1.5 Related Documents

This section briefly describes how these *Guidelines* relate to existing standards and post-earthquake assessment documents.

- The FEMA 306/307/308 series *on* evaluation and repair of earthquake-damaged concrete and masonry wall buildings (FEMA, 1999 a,b,c). These *Guidelines* update and supersede the FEMA 306/307/308 series with respect to evaluation and repair of earthquake-damaged concrete wall buildings. These *Guidelines* also extend the scope beyond the FEMA 306/307/308 series to include concrete frame systems. Guidance for masonry wall buildings may be added in future editions of these *Guidelines*.

- ASCE/SEI 41, *Seismic Evaluation and Retrofit of Existing Buildings*. ASCE/SEI 41 contains provisions for seismic retrofit that can be applied for buildings evaluated in accordance with these *Guidelines*. These *Guidelines* include procedures for determining the necessity of seismic retrofit of an earthquake-damaged building, but they do not include procedures for designing the seismic retrofit. The *Guidelines* also refer to ASCE/SEI 41 for guidance on analysis and modeling parameters. At the time of this writing, the latest version of ASCE/SEI 41 was published in 2023, but future editions can and should be used.

- ATC-20-1, *Field Manual: Postearthquake Safety Evaluation of Buildings, Second Edition*. ATC-20-1 provides rapid and detailed procedures for evaluating earthquake-damaged buildings and posting them with INSPECTED (green), RESTRICTED USE (yellow), or UNSAFE (red) placards. These placards indicate the occupancy that is permitted, given the current condition of the building. While not required, the results of an ATC-20-1 evaluation, if available, may be useful in informing preliminary inspections involved in the assessments in these *Guidelines*.

- *International Existing Building Code*. The IEBC covers repair, alteration, addition, and change of occupancy for existing buildings, and provides criteria for retrofit and repair. These *Guidelines* adopt concepts from the IEBC to ensure that assessments using the procedures herein can be interpreted in the context of the IEBC, where adopted. Certain one and two-family dwellings may comply with the *International Residential Code* (ICC, 2024b) rather than the IEBC. Nevertheless, these *Guidelines* are considered applicable to such buildings, and dwellings are not excluded. At the time of this writing, the most recent version of the IEBC is the 2024 edition. Because the IEBC is published every three years, users should be cognizant of changes in the IEBC that may affect how these *Guidelines* should be applied.

1.6 Definitions and Notation

These *Guidelines* use concepts and language that are familiar to many practicing engineers working in the field of earthquake-resistant design and assessment. These *Guidelines*, however, introduce some terms whose definitions are particular in their use in this document, or not in common use. Such terms are defined in the Glossary. To the extent practicable, these *Guidelines* use familiar symbols and notation. Symbols and notation are listed in the Notation appendix.

1.7 Limitations

The procedures and criteria herein have been developed based on the current state of the knowledge on the behavior of building structures and structural components subjected to earthquake ground shaking. This knowledge will expand over time, and the assessment, repair, and retrofit procedures described herein should be adjusted over time to reflect new knowledge.

The interpretation of damage and the performance of buildings subjected to earthquakes necessarily requires engineering judgment. These *Guidelines* provide a framework and specific methods through which an engineer can apply experience and formulate judgments on the effects of earthquake damage and methods to address the damage. The engineering procedures contained in these *Guidelines* are intended for application by engineers with experience in seismic design and assessment and should not be applied by non-engineering personnel (including, but not limited to, inspectors, owners, contractors, insurance adjusters, and claims managers).

These *Guidelines* assess earthquake damage in the context of its implications for building seismic performance. However, the absence of damage does not indicate that a building in its pre-earthquake condition does not have earthquake-performance deficiencies. The repair strategies identified herein restore an earthquake to its pre-earthquake condition. Other documents, such as ASCE/SEI 41, should be used to identify deficiencies that existed before the earthquake.

At the present time, the detailed criteria herein are applicable to reinforced concrete structures. Until these *Guidelines* are extended to other structural materials and systems, other resources are recommended. These include the FEMA 306/307/308 series for masonry structures and FEMA 352, *Recommended Postearthquake Evaluation and Repair Criteria for Welded Steel Moment-Frame Buildings* (FEMA, 2000), for steel moment-frame buildings.

Chapter 2: Overview

2.1 Post-Earthquake Assessment Process

The request or requirement for assessment of a building after an earthquake can be made by an individual owner or by an Authority Having Jurisdiction. These *Guidelines* do not specify the conditions under which such an assessment is needed. The assessment in these *Guidelines* focuses on Performance-Critical Damage, which is consistent with policies regarding repair and retrofit in place in many jurisdictions. The relevant policies in the building's jurisdiction should be reviewed to confirm that the focus on Performance-Critical Damage is sufficient.

Once these determinations are made, these *Guidelines* describe a process for post-earthquake assessment and repair and retrofit evaluation illustrated in Figure 2-1. This process involves both identification of earthquake damage through inspection and analysis and evaluation of earthquake damage to determine building repair outcomes.

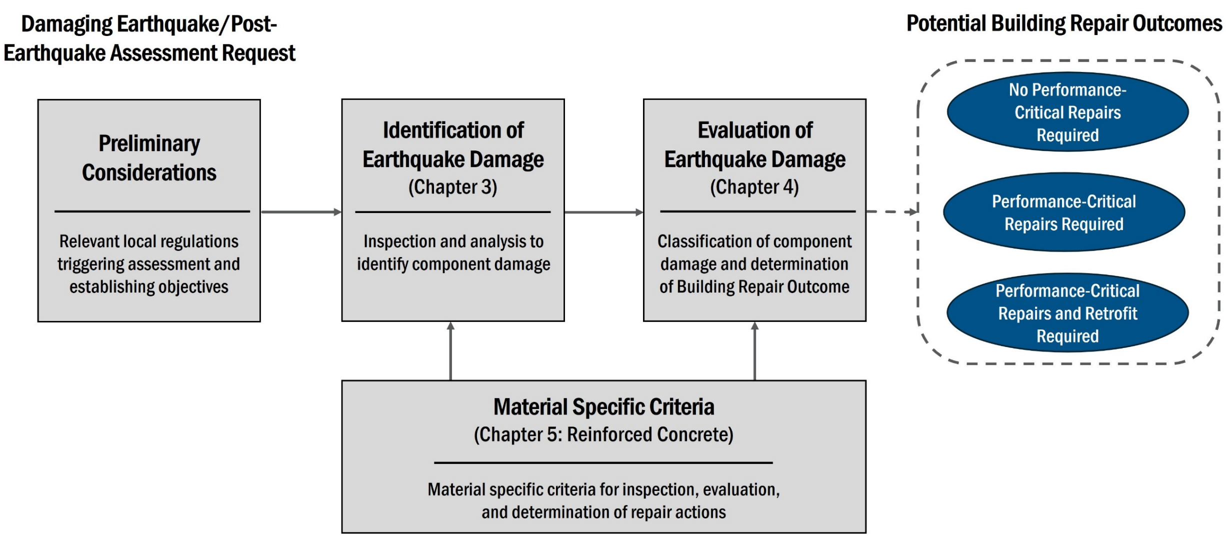


Figure 2-1 Flowchart of process for post-earthquake building assessment and repair and retrofit evaluation.

2.1.1 Identification of Earthquake Damage

The *Identification of Earthquake Damage* (Chapter 3) involves visual inspection and analysis, as shown in Figure 2-2. This process begins with a preliminary inspection where the engineer collects data on the damaging earthquake and the original building construction and conducts a visual inspection to identify earthquake damage (Section 2.2.3.1). Detailed visual inspections are then performed (Section 2.2.3.2). Depending on the circumstances, structural analyses (Section 2.2.7) may be performed to guide the detailed inspections and aid in the interpretation of the earthquake damage. Where component damage is strongly suspected but is not evident from detailed visual inspections, intrusive inspections may be required (Section 2.2.3.3). The goals of the inspections and analyses are to identify the *Damage States* and associated *Damage Classes* of the structural components (Section 2.2.4.2).

2.1.2 Evaluation of Earthquake Damage

In the next phase, *Evaluation of Earthquake Damage* (Chapter 4), the documented earthquake damage is classified as to its severity to determine repair and retrofit needs. This process is illustrated in Figure 2-3. This involves documenting the component Damage Classes (Section 2.2.4.2), an outcome of *Identification of Earthquake Damage* (Chapter 3) that establishes components with Performance-Critical Damage. The determination of component Damage Classes (Section 2.2.4) is based on Damage States, which are determined by a combination of *Visual Damage States* (VDS) and other factors (Section 2.2.5).

The *Building Repair Outcome* (Section 2.2.6) indicates whether the structure needs performance-critical repair or repair and retrofit. Determination of the Building Repair Outcome is intended to be consistent with the requirements of the IEBC. As shown in Figure 2-4, if there is no Performance-Critical Damage, repairs may be limited to actions that restore appearance and durability. If Performance-Critical Damage is identified, analysis may be required to determine whether the building is compliant with acceptable building codes and whether the building has sustained *substantial structural damage* or *disproportionate earthquake damage*. In many cases, the necessary action may be limited to performance-critical repair, i.e., repairing damaged structural components to restore their pre-earthquake strength and deformation capacity. However, as described in Chapter 4, in some cases this process may indicate that the building also requires retrofit.

2.1.3 Material-Specific Chapters

Material-specific procedures in Chapter 5 (Reinforced Concrete), and additional chapters to be added in future editions for structures composed of other materials, provide specific criteria and databases of damage component information that are necessary for the identification and evaluation of earthquake damage, and to guide how repair and retrofit can be implemented.

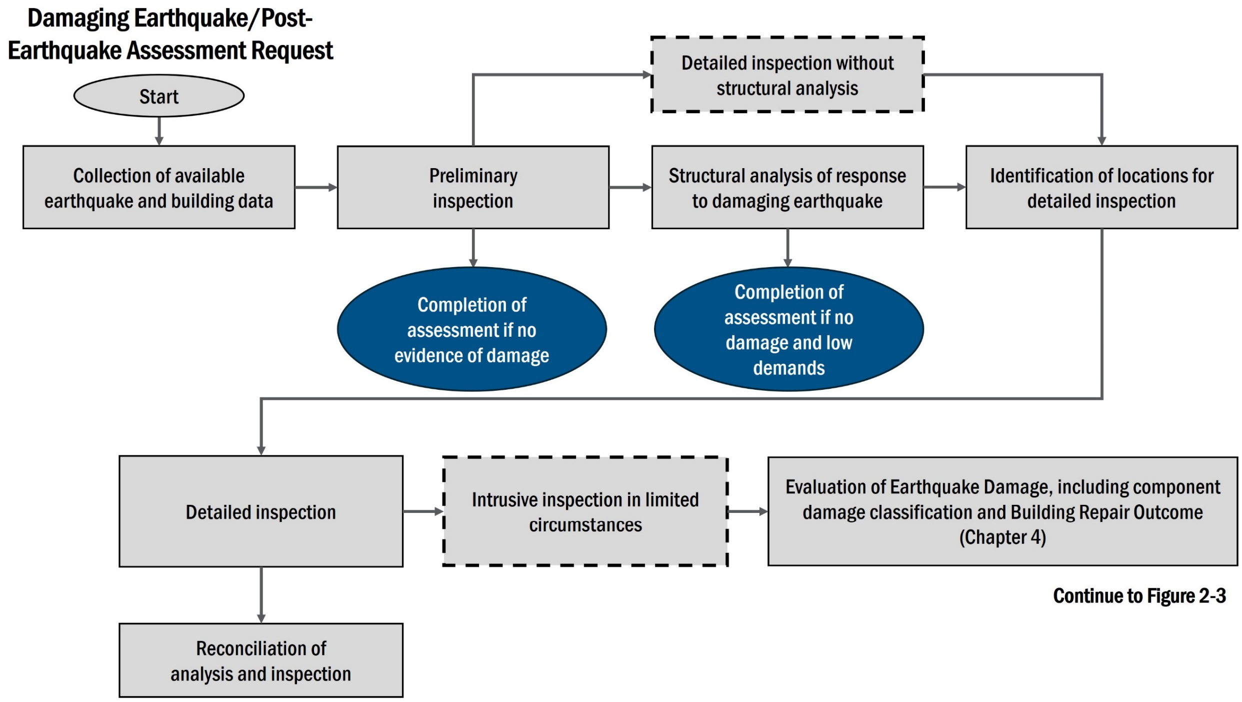


Figure 2-2 Process for *Identification of Earthquake Damage* (Chapter 3). Dashed lines indicate steps that might not be taken in all assessments.

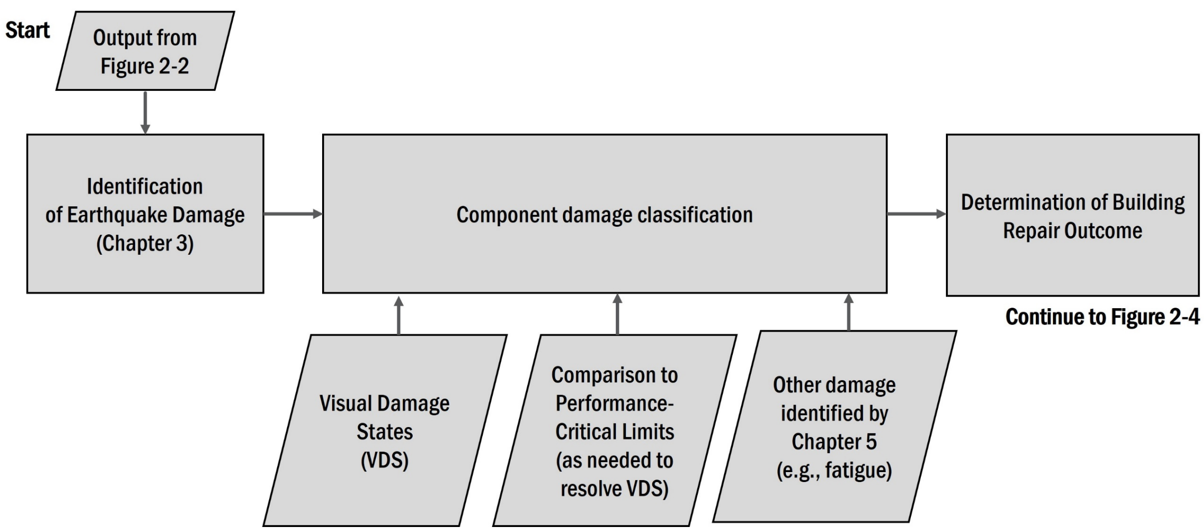


Figure 2-3 Process for *Evaluation of Earthquake Damage*: Damage Classification (Chapter 4).

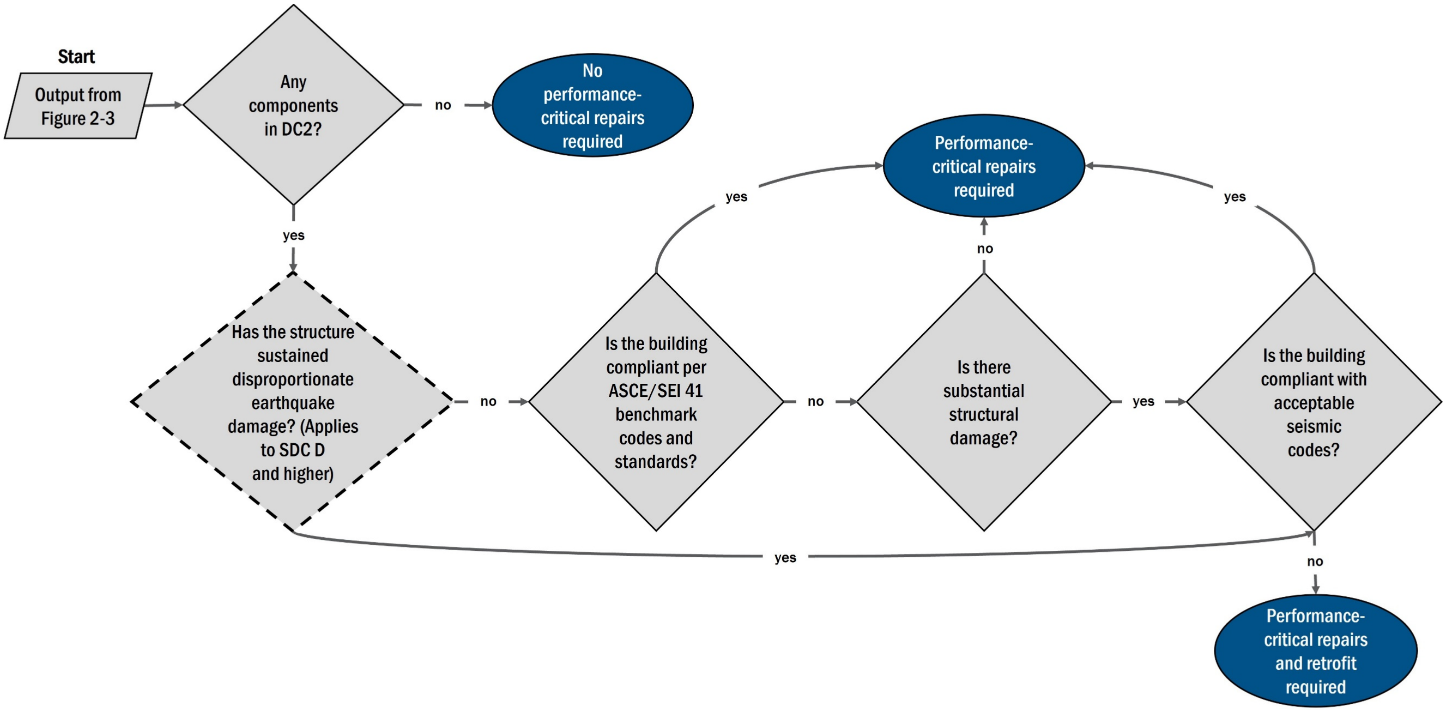


Figure 2-4 Process for *Evaluation of Earthquake Damage*: Building Repair Outcome (Chapter 4). Dashed lines indicate steps that might not be taken in all assessments. Components in Damage Class 2 (DC2) have sustained Performance-Critical Damage, as described in Section 2.2.4.

2.2 Key Concepts

The primary objective of these *Guidelines* is to identify, evaluate, and determine necessary repair actions for earthquake-damaged structural components in buildings. Component damage is classified as being performance-critical or not performance-critical. When Performance-Critical Damage is identified, the building is evaluated to determine whether repair or repair and retrofit are needed. These *Guidelines* also identify repair strategies that can be used to address earthquake damage. The following sections describe key concepts used throughout these *Guidelines*. Familiarity with these concepts is necessary for effective application of the procedures in these *Guidelines*.

2.2.1 Structural Systems, Elements, and Components

The assessment of damage to a building requires the engineer to develop an understanding of the way in which the building supports gravity loads, resists earthquake actions, and accommodates earthquake-induced displacements. For this purpose, it is helpful to conceive of the overall building structure as an assembly of elements (see Figure 2-5). An *element* is a vertical or a horizontal portion of a building that acts to resist lateral or vertical loads or both. Example vertical elements in buildings are structural walls and beam-column frames. Example horizontal elements are reinforced concrete diaphragms. The building will also have a foundation that transmits gravity and earthquake-related forces to the supporting soil. Some of these elements may have been designed as part of a lateral-force-resisting system or they may have been designed primarily to support gravity loads. These *Guidelines* require that all parts of the structure that resist lateral forces or support gravity loads be considered when assessing earthquake damage.

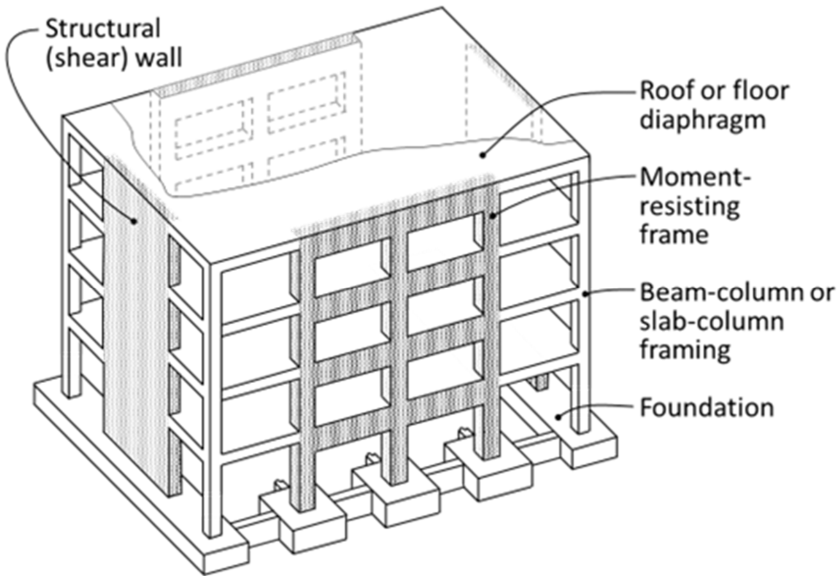


Figure 2-5 Illustration of reinforced concrete structural systems (credit: Moehle, 2015).

Elements are assemblies of individual *components* such as beams, slabs, and columns. Figure 2-6 illustrates the relationship between the global structure, one of its vertical elements (Wall Element A), and the components that comprise that vertical element. The overall performance of the structural system is an aggregation of the performance of its components. These *Guidelines* assess earthquake damage by the type and severity of damage that occurs in each of the components.

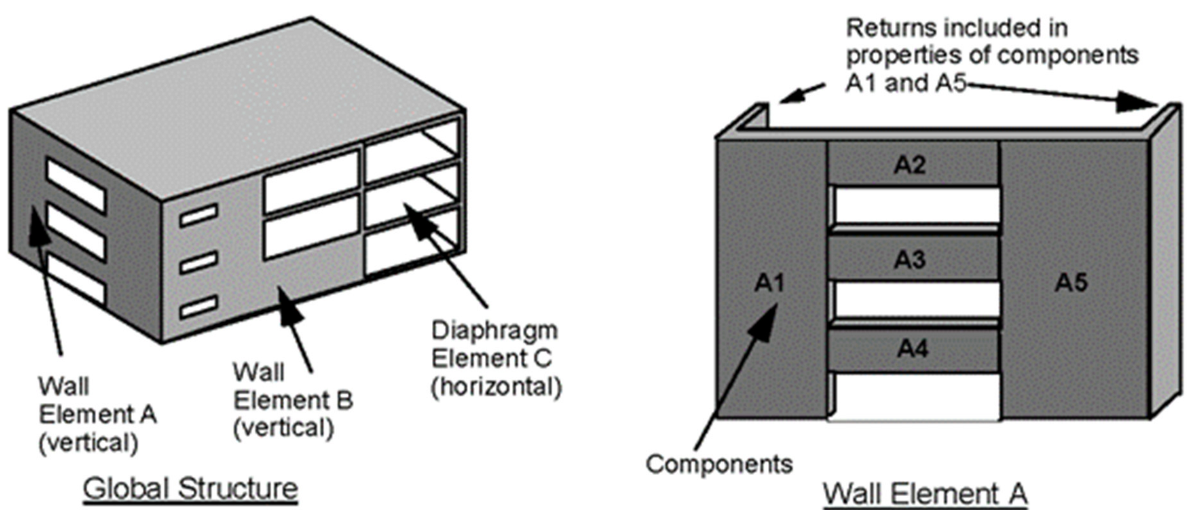


Figure 2-6 Illustration of global structure, its elements, and components of an element (credit: FEMA, 1999a).

2.2.2 Performance-Critical Damage and Repair

A component has experienced Performance-Critical Damage if the damaging earthquake has imposed demands that caused the component and, therefore, the structural system to lose strength, deformation capacity, or both. This damage is critical to the future performance of the building. Performance-Critical Damage to components occurs if demands exceed the point of initial of component strength loss. This point is shown in Figure 2-7, with reference to an example set of test data. Determination of whether Performance-Critical Damage has occurred is made through classification of component Damage Classes (Section 2.2.4.2; Section 4.3) with reference to Visual Damage States (Section 2.2.5.1) and, where needed, Performance-Critical Limits (Section 2.2.5.2). Chapter 5 also defines a limited number of additional types of Performance-Critical Damage that are not based on the initiation of component strength loss (e.g., reinforcing steel fatigue damage in reinforced concrete components, Section 5.6). Repair of components with Performance-Critical Damage is required to restore the pre-earthquake strength and deformation capacity in order to restore the pre-earthquake performance of a building in terms of drift control and collapse risk.

> ***Commentary:*** *The selection of the point of initiation of component strength loss or building lateral strength loss as the critical point is based on extensive study of past earthquake damage, review of experimental data, and analytical studies (See, e.g., Murray et al., 2022; Opabola et al., 2023; Safiey et al., 2022; Shah, 2021). When components or buildings are subject to demands beyond this point, future earthquake performance is substantially impaired, indicating a loss of strength, deformation capacity, or both relative to the building's pre-earthquake condition. As a result, without repair, the building would sustain amplified drift demands with higher probabilities of collapse and a decrease in safety relative to its pre-earthquake condition.*
>
> *The absence of Performance-Critical Damage does not necessarily mean that the building will perform well in future earthquakes; rather, this indicates only that the building's future performance capability has not been reduced by the earthquake shaking that led to the post-earthquake assessment. Likewise, repair of Performance-Critical Damage restores components and the building to their pre-earthquake strength and deformation capacity. These repairs do not address pre-existing building earthquake vulnerabilities.*
>
> *A component envelope is shown in Figure 2-7, which defines the force-displacement response of an example component and illustrates the point of initiation of component strength loss. This envelope and the corresponding point associated with Performance-Critical Damage is defined based on the component action that contributes to building lateral strength. For example, for a flexure-critical beam or column, the point of initiation of component strength loss is defined in terms of a moment (and rotation) demand and generally corresponds to bar buckling.*
>
> *This point of initiation of component strength loss is generally less than the ASCE/SEI 41* a *or* d *values (ASCE, 2023), which represent the point at which 20% lateral strength has been*

lost. For flexural-dominated reinforced concrete components, at 20% component strength loss, both bar buckling and fracture have typically occurred.

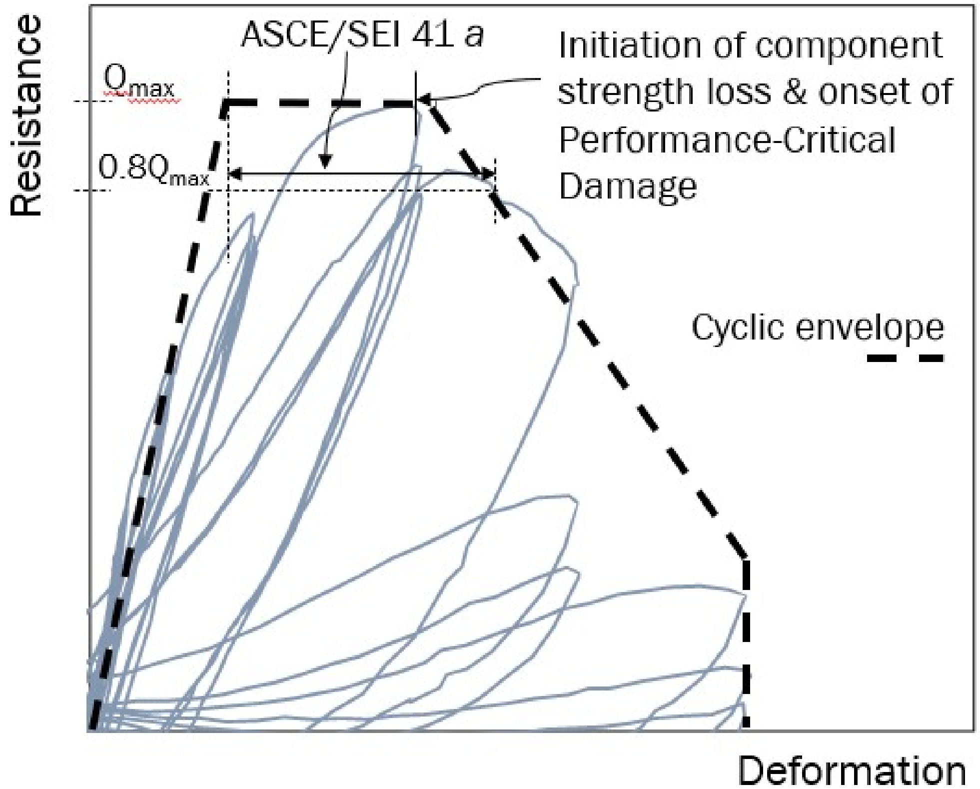


Figure 2-7 Illustration of component test force-deformation response data showing definition of cyclic envelope and point of initiation of component strength loss. This point is also compared to the ASCE/SEI 41 value for *a*. The experimental data are from a reinforced concrete column tested by Sezen (2002), with resistance measured by shear force and deformation by drift ratio.

2.2.3 Inspection

Inspection (Section 3.5 to Section 3.8) is the part of the post-earthquake assessment process that is conducted at the building and involves visual observation. This visual observation may necessitate removal of nonstructural and structural elements. Three different types of inspection are defined in this document, which are progressively more involved.

2.2.3.1 PRELIMINARY INSPECTION

Preliminary inspection (Section 3.3) involves a site visit with visual inspection of the exterior and interior of the building. Preliminary inspection could include nondestructive removal of nonstructural finishes (e.g., ceiling tiles, access panels). This site visit should be preceded or accompanied by a collection and analysis of earthquake data, and review of existing building data. As much existing building data as possible should be collected to facilitate the subsequent assessment, although this may be very limited or nonexistent. The preliminary inspection can be used to classify the structure as undamaged if the conditions described in Section 3.3.5 are met.

2.2.3.2 DETAILED INSPECTION

Detailed inspection (Section 3.6) is a visual inspection that exposes the surface of structural components. Detailed inspection may involve destructive removal of nonstructural finishes (e.g., gypsum board wall and ceiling finishes, fireproofing). Locations of possible damage for detailed inspection (Section 3.5) are informed by preliminary inspection, conditions always requiring inspection, structural irregularities, and mechanisms analysis and are based on earthquake demands calculated from structural analysis.

Inspection Indicators

Inspection indicators (Section 3.5.5) may be used to identify which components require detailed inspection based on calculated earthquake demands. If structural analysis has been conducted, the demand in the component from structural analysis is compared to the calculated value of the inspection indicator. Inspection indicators depend on the component, condition, and behavior mode. They are defined such that there is a low likelihood of missing Performance-Critical Damage during detailed inspection. Section 3.5.5 defines the calculation of inspection indicators, which depends on the level of uncertainty in the available information and structural analyses, as well as the *Performance-Critical Limit* (Section 2.2.5.2) determined for a component of interest.

2.2.3.3 INTRUSIVE INSPECTION

Intrusive inspection (Section 3.8) is an interrogation of structural components carried out in a limited number of instances that can involve removal of structural materials to expose elements of the component that are not otherwise visible, nondestructive testing, or other evaluation methods depending on the structural material, structural element, and the expected loads. Guidance for what intrusive inspection may entail and situations when it may be needed are provided in Chapter 5.

> *Commentary: Removal of structural materials can impact future performance or increase complexity of repair and should be avoided where possible. Such intrusive inspections are typically not required by these* Guidelines *except in specific conditions identified in Chapter 5.*

2.2.4 Component Damage States and Damage Classes

2.2.4.1 DAMAGE STATES

Component Damage States (DS) refer to specific points on the envelope of a component's cyclic force-deformation response. These Damage States are shown on the idealized component envelopes in Figure 2-8 and defined here.

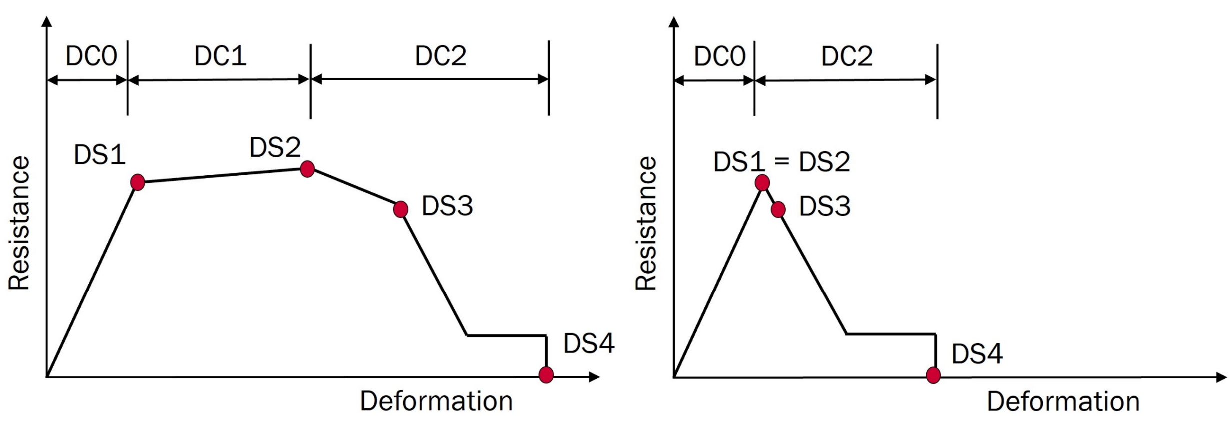
- DS1: end of essentially elastic response

- DS2: initiation of component strength loss
- DS3: 20% drop in post-peak component strength
- DS4: gravity load failure

Commentary: *DS2 defines the boundary between components that exhibit Performance-Critical Damage and those that do not exhibit Performance-Critical Damage. Performance-Critical Damage requires performance-critical repairs. DS2 is the primary Damage State of interest in these* Guidelines*.*

DS1 represents the end of essentially elastic behavior. It corresponds to distributed yielding, rather than first yield. DS2 represents the point of initiation of component strength loss. This point corresponds to the point of peak strength. DS3 represents the point of 20% strength loss from the peak strength and is consistent with the definition of a and d values in ASCE/SEI 41. DS4 is not a focus of these Guidelines and *represents the point of loss of gravity-load-carrying capacity. DS4 corresponds to b or e values in ASCE/SEI 41. Databases with detailed information related to these damage states were developed for each type of component (Section 2.2.5.1).*

Some components have significant inelastic deformation capacity prior to initiation of component strength loss, and therefore DS1 and DS2 may occur at substantially different deformations (Figure 2-8a). For these components, which are termed "deformation-controlled," DS2 is defined in terms of a measure of deformation. Other components have little or no inelastic deformation capacity, and DS1 and DS2 coincide at the point of peak strength (Figure 2-8b). For these "force-controlled" components, DS2 is defined in terms of a measure of force.



(a) **(b)**

Figure 2-8 Component envelopes defining component Damage States (DS) and component Damage Classes (DC) for components (a) with capacity for significant inelastic deformation and (b) with little or no inelastic deformation prior to the initiation of component strength loss.

2.2.4.2 DAMAGE CLASSES

Components are classified in component Damage Classes (DC) based on observed damage and the Damage State definitions, as described in Table 2-1. These Damage Classes encompass a range of observed damage (and a range of responses on the component force-displacement envelope) with shared damage characteristics and repair outcomes.

Table 2-1 Component Damage Classes

Damage Class	Relation to Component Damage States	Damage Description
DC0	Prior to DS1; Essentially elastic response	No Performance-Critical Damage; unlikely to require cosmetic, durability, or serviceability repairs
DC1	Between DS1 and DS2; Inelastic response without component strength loss	No Performance-Critical Damage unless indicated by other material-specific checks (e.g., fatigue damage), but cosmetic, durability, or serviceability repair may be warranted
DC2	Past DS2; Inelastic response with component strength loss	Performance-Critical Damage

> ***Commentary:*** *DC2 is the primary Damage Class of interest in these* Guidelines *because, if a component is in DC2, it requires performance-critical repairs to restore its pre-earthquake strength and deformation capacity. The definition of DC2 reflects evidence of impaired future earthquake performance of components that have been subjected to demands that have gone beyond the point of initiation of component strength loss. For buildings with components in this Damage Class, future earthquake performance in terms of drift demands and collapse risk may be impaired (ATC, 2021 a,b).*

Components in DC2 contribute to a loss of building lateral capacity. These components may also exhibit loss of gravity-load-carrying capacity. Components that carry substantial gravity loads and that may lose gravity-load-carrying capacity shortly after losing lateral-load-carrying capacity are classified as gravity-essential components. Gravity-essential components whose damage is classified in DC2 need to be considered in the assessment of substantial structural damage.

2.2.5 Determination of Component Damage Classes

The damage observed during inspection is used to determine if any components are in DC2 (i.e., if Performance-Critical Damage has occurred). To determine component Damage Classes, components are classified by their geometry, condition, and behavior mode. Damage Classes are determined primarily with reference to Visual Damage States, as described in Section 2.2.5.1. If the Visual Damage States are not conclusive, the Performance-Critical Limits (Section 2.2.5.2) can be used in conjunction with structural analysis results to inform the determination of Damage Class. In limited cases, other indicators of Performance-Critical Damage (Section 2.2.5.3) may also need to be evaluated.

2.2.5.1 VISUAL DAMAGE STATES

Observed damage to components is classified by comparing the component damage observed during inspections with key visual damage features (referred to as *Visual Damage States*) found in similar components that have experienced Performance-Critical Damage. Chapter 5 provides a list of key damage features consistent with Performance-Critical Damage for different components, which may be useful to help guide the inspecting engineer. Furthermore, electronic VDS databases, provided as companions to Chapter 5, include photographs of components from laboratory tests corresponding approximately to states DS1, DS2, DS3, and DS4 (Figure 2-9). See Section 5.5.2 and Appendix B for more information about the available VDS databases. By comparing the VDS photographs with the observed state of an earthquake-damaged component in the field, the inspecting engineer can determine the Damage Class for the component. If the component is in DC2, the component requires performance-critical repair.

Chapter 5 also provides guidance on how to select appropriate laboratory experiments for the purpose of making this comparison. If the review of Visual Damage States is not definitive, the Performance-Critical Limits (Section 2.2.5.2) are used.

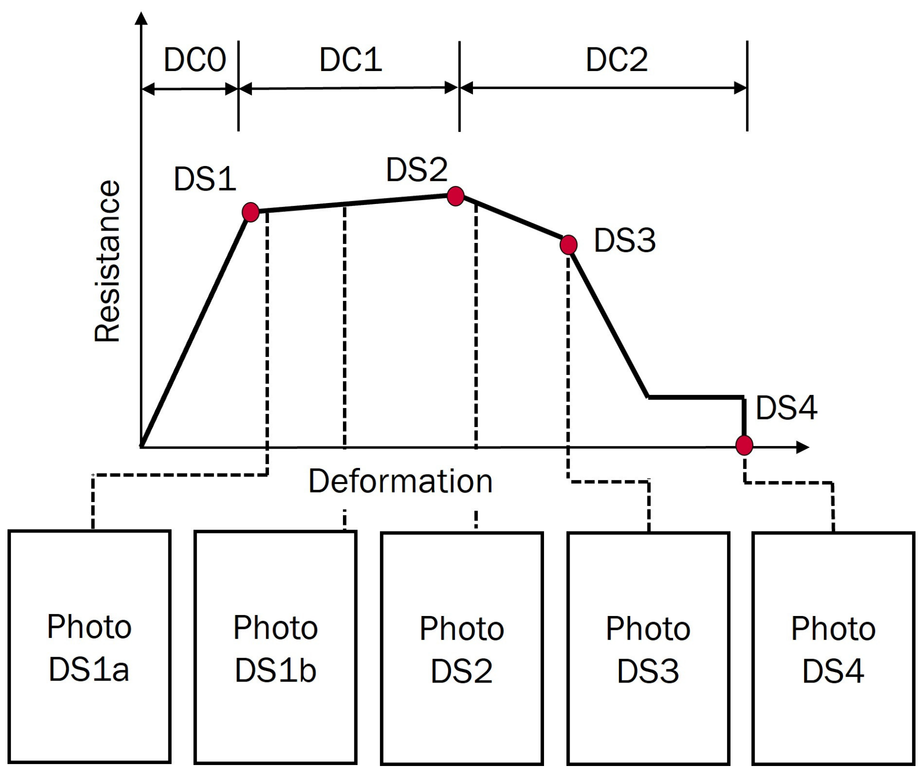


Figure 2-9 Illustration of Visual Damage States, used to determine component Damage Classes. Photographs are provided in electronic VDS databases to help in determining the Damage Class of an earthquake-damaged component.

2.2.5.2 PERFORMANCE-CRITICAL LIMITS

The Performance-Critical Limits (Section 5.4) are quantitative metrics that depend on the component characteristics, providing a median estimate of the deformation (or force) at DS2. These can be compared with deformation (or force) demands from structural analysis to determine if the component has likely exceeded DS2 during the damaging earthquake. Performance-Critical Limits are intended to support or clarify component damage classifications in cases where the conclusion based on the observed damage and the Visual Damage State databases is unclear.

Chapter 5 defines Performance-Critical Limits by component type, characteristic, and behavior. These may be force- or deformation-controlled (Section 5.4).

> Commentary: *The values for the Performance-Critical Limits have been determined from databases of laboratory tests. The limits may be defined in terms of deformation (e.g., plastic rotation or chord rotation) or force demand, as appropriate for the critical action in the component, as shown in Figure 2-10. In most cases, they are defined in terms of fractions, η, of the ASCE/SEI 41 modelling parameters a and d, although in isolated cases revised values of a and d are defined. The a and d values provide measures of deformation at DS3 (20% drop in resistance), so the Performance-Critical Limits are smaller.*

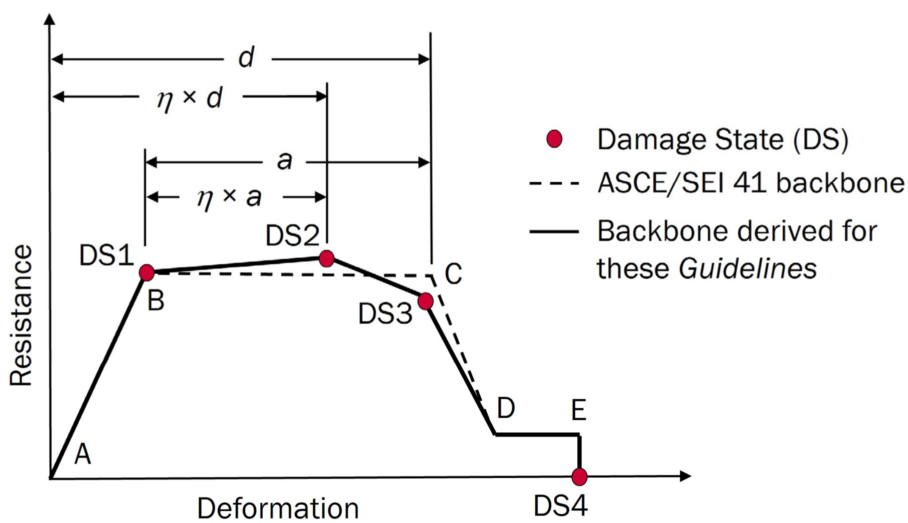


Figure 2-10 Generalized force-deformation relationship for components.

2.2.5.3 OTHER INDICATORS OF PERFORMANCE-CRITICAL DAMAGE

Other indicators of Performance-Critical Damage, which cannot be detected by the Visual Damage States or the Performance-Critical Limits may also need to be checked. This applies, for example, to structural components that can experience cyclic fatigue, which depends on both the intensity and duration of the damaging earthquake motion. Guidance for checking these other conditions is provided in Chapter 5. Specifically, Section 5.6 includes procedures to check for cyclic fatigue damage to steel reinforcement in concrete structures.

2.2.6 Building Repair Outcomes

2.2.6.1 REPAIR

Performance-critical repairs refer to repairs undertaken to restore structural components to their pre-earthquake condition in terms of strength and deformation capacity.

Repairs to address cosmetic damage, improve durability, or address serviceability are generally permitted, either as the sole repair objective where performance-critical repair is not needed or in conjunction with those repairs.

> ***Commentary:*** *Details of performance-critical repair options are provided in Chapter 5, e.g., Section 5.9.3 and Table 5-9. Performance deficiencies of the building's pre-earthquake condition are not addressed by performance-critical repairs. Other types of repair may include patching, sealing, and painting to improve cosmetic, durability, or serviceability issues. These types of repairs typically do not qualify as performance-critical repairs because they may or may not restore strength or deformation capacity.*

2.2.6.2 RETROFIT

Retrofit refers to upgrades to structural components or the structural system that improve seismic performance relative to the pre-earthquake condition.

2.2.6.3 BUILDING REPAIR OUTCOME

A Building Repair Outcome (Section 4.4) is determined in accordance with Figure 2-4. The possible Building Repair Outcomes are: (1) no performance-critical repairs required, (2) performance-critical repairs required, and (3) performance-critical repairs and retrofit required.

If all components are in DC0 or DC1, the building repair outcome is "no performance-critical repairs required."

The building requires performance-critical repairs if any component is classified as DC2. If Performance-Critical Damage is identified, it is necessary to determine whether the building also requires retrofit following the process shown in Figure 2-4. This process to determine if retrofit, as well as repair, is required is consistent with the *International Existing Building Code* and considers: (1) compliance with acceptable seismic codes (Section 4.4.1); (2) whether the building sustained disproportionate earthquake damage in the damaging earthquake (Section 4.4.2), and (3) whether the building sustained substantial structural damage in the damaging earthquake (Section 4.4.3).

> ***Commentary:*** *The scope of repairs and retrofit identified per these* Guidelines *is governed by applicable codes and regulations and the Authority Having Jurisdiction. An Authority Having Jurisdiction may have a repair and retrofit requirement that differs from the International Existing Building Code.*

During the process of assessing damage and determining repairs, the owner or engineer may wish to examine possible upgrades to the building that could accompany the performance-critical repair actions. Voluntary upgrades or retrofits should be evaluated using ASCE/SEI 41 and other policies adopted by the Authority Having Jurisdiction. The decision to voluntarily retrofit depends on a number of interrelated factors including the severity of the damaging ground motion, the acceptability of the performance characteristics of the building after the damaging earthquake, the acceptability of performance characteristics of the building before the damaging earthquake, and nonseismic considerations (e.g., maintenance, programmatic issues). Voluntary and triggered mitigation actions may then require other work, such as related to accessibility, as determined by the Authority Having Jurisdiction, and these topics are not addressed here.

Substantial structural damage represents significant damage to the structure's lateral capacity, raising questions about the building's future performance. Disproportionate earthquake damage exists "where a building has significant damage in even a very small earthquake. This damage is an indicator of severe damage, possibly collapse in a larger event" (IEBC, 2024).

2.2.7 Structural Analysis

The procedures in these *Guidelines* often involve the use of *structural analyses*, including the definition of one or more simulation model(s) of the building. During the Identification of Earthquake Damage (Chapter 3), structural analyses may be used to: (1) guide detailed inspection by estimating structural component force or deformation demands in the damaging earthquake, and (2) (if necessary) help identify component Damage Classes (Section 2.2.4.2) where Visual Damage States (Section 2.2.5.1) are inconclusive. During Evaluation of Earthquake Damage (Chapter 4), structural analysis may be used to determine the Building Repair Outcome (Section 2.2.6). Where seismic retrofit is indicated, structural analysis may also be needed to design the seismic retrofit. The same simulation model or analysis method may or may not be appropriate for each of these needs.

2.2.7.1 GENERAL CONSIDERATIONS

These *Guidelines* do not prescribe how structural analysis should be done. A variety of structural analysis approaches (e.g., using linear or nonlinear models) may be appropriate depending on the condition of the building. The selection of the analysis model and procedure(s) should consider the available ground motion information, level of damage in the building, the anticipated needs for analysis in carrying out these *Guidelines*, and the time available to develop and complete the analysis.

The structural analyses in these *Guidelines* are intended to represent realistic or expected response. As such, key quantities in the analysis, including material properties and gravity loading, should be defined in terms of expected values, rather than lower-bound values.

Commentary: Existing standards, guidelines, and design briefs provide guidance on how to develop and verify a structural analysis model. Among these, ASCE/SEI 41 contains provisions for the analysis of existing buildings, including general analysis requirements, procedures for selecting among various analysis methods, specific analysis requirements, component modeling parameters, and procedures for developing alternative modeling parameters. Those procedures are applicable for use with these Guidelines*, except as noted.*

Some analysis requirements in ASCE/SEI 41 are intended to produce a conservative value of deformation or force demands. In contrast, the goal of the analysis in these Guidelines *is to obtain a best estimate of demands on a building and the damage incurred. Therefore, component strengths should be determined using expected material properties with no strength reduction factors. Gravity loads should be based on best estimate loads. Similarly, torsional amplification requirements in ASCE/SEI 41 will generally overstate the effect of any eccentricity of the center of mass relative to the center of lateral rigidity, artificially increasing demands; therefore, torsional amplification need not be considered.*

2.2.7.2 STRUCTURAL ANALYSIS OF BUILDING RESPONSE TO DAMAGING EARTHQUAKE

Structural analysis of building response to the damaging earthquake is used to inform inspections and component damage classifications. In particular, demands estimated from structural analysis can be compared to inspection indicators and Performance-Critical Limits to inform determination of inspection locations and, if necessary, identification of Performance-Critical Damage.

The analysis procedure used to determine building response to damaging earthquake shaking may involve use of a linear or nonlinear analysis model subjected to a response spectrum or acceleration history representation of the earthquake loading. Guidance for performing this analysis, including seismic demand representation of the damaging earthquake and calculation of critical demand parameters, is provided in Section 3.4.

Commentary: In deciding on what type of analysis to use for evaluating the building response and component demands to the damaging earthquake (Section 3.4), the engineer should consider what type of analyses may be subsequently required (Chapter 4) for evaluating the impact of the damage on the lateral-load-carrying capacity of the building and designing earthquake repairs and retrofits. For example, if one anticipates the need to use nonlinear analyses during the Evaluation of Earthquake Damage (Chapter 4), this may be an incentive for also using nonlinear analyses for the Identification of Earthquake Damage (Chapter 3).

2.2.7.3 ANALYSIS OF REDUCTION IN LATERAL-FORCE-RESISTING CAPACITY

The need to determine disproportionate earthquake damage and substantial structural damage in Section 4.4.2 and Section 4.4.3 may lead to the assessment of the reduction of the lateral-force-resisting capacity caused by the damage.

The determination of the reduction of lateral-force-resisting capacity should consider the same analysis approach used to assess the pre-damage condition, with modification to the properties of components in DC2 to reflect this damage. More details are provided in Section 4.4.4.

> *Commentary: The goal of these analyses is to provide an assessment of the global lateral capacity of the structure and the reduction in capacity associated with the earthquake damage.*

2.2.7.4 OTHER ANALYSIS

There are a variety of other potential uses for structural analysis in these *Guidelines*. These include: (1) analysis to determine compliance with acceptable seismic codes; (2) analysis to design a repair; (3) analysis to design a retrofit; and (4) analysis to assess alternative performance objectives (e.g., serviceability). In some cases, it may be necessary to modify the analysis to represent the condition of the damaged or repaired components. Chapter 4 provides details on strength and deformation capacity modification factors for repaired components. Chapter 5 provides estimates of stiffness modification factors for damaged and repaired components.

2.3 Reporting

Where required by the Authority Having Jurisdiction, building owner, or other stakeholder, a written report should be prepared to document the process and key findings and recommendations of the investigation. Reports should include the following information:

1. Background information on the building and preliminary inspection, including: (1) summary of the structural system and its expected behavior under the damaging earthquake; (2) a narrative, drawings, and photographs of observed damage to structural and nonstructural components; (3) notes of any significant discrepancies between the expected and observed condition and behavior of the building; and (4) support for classification of structure as undamaged as applicable.
2. Detailed visual inspections and resulting Visual Damage States (VDS) including: (1) summary of how the detailed visual inspections were conducted, including details of how the locations of the detailed visual inspections were determined (and reduced, as applicable), and (2) a narrative, drawings, and photographs of the observed Visual Damage States.
3. If structural analysis is conducted, details of the structural analysis including: (1) the seismic demand representation; (2) the type of structural analysis and summary of key modeling parameters; (3) summary of the structural components and their Performance-Critical Limits; (4) summary of the calculated component earthquake demands and resulting damage classifications; (5) calculation of inspection indicators and locations requiring detailed visual inspection; and (6) support for classification of the structure as undamaged as applicable.
4. Where appropriate, description and reconciliation of significant differences between the damage classifications as inferred from the structural analyses and detailed visual inspections, including:

(1) whether intrusive inspections were warranted and performed, and (2) a narrative, drawings, and finding from any intrusive inspections.

5. Compilation of component damage classifications resulting from the investigation of earthquake damage.
6. Determination of the Building Repair Outcome per Chapter 4.

> *Commentary: The triggering requirements for preparation of a written report are similar to those provided in ASCE/SEI 41. The report serves to communicate the results of the investigation to the relevant stakeholders and document the process including any assumptions. Stakeholders may include the Authority Having Jurisdiction, client, building owner, tenants, financial lenders, and insurers. The need for a report will vary by circumstance. An Authority Having Jurisdiction may require reports to be prepared to facilitate consistent and expedient reviews of multiple buildings. The level of detail provided in item 2 to item 5 may be valuable in the event of a disagreement between stakeholders or other consultants. Not all of the listed report items may be applicable, depending on the extent of damage identified during the investigation phase.*

Chapter 3: Identification of Earthquake Damage

3.1 Scope

This chapter provides in-depth guidance and essential commentary on the process of identifying earthquake damage through inspection and structural analysis. The process begins with a preliminary inspection, in which data are collected on the earthquake and the building, and a site visit, in which initial data are collected regarding building damage. For buildings where detailed inspection of the structural components is impeded by architectural enclosures or other obstructions, or where damage is not otherwise obvious, a structural analysis of the building under the damaging earthquake shaking may be performed to help identify areas within the building that may have damage. A subsequent detailed inspection is then carried out. A reconciliation of the inspection and analysis results may be needed to reconcile any discrepancies between observations and calculations and to better refine understanding about the damaged condition of the building. Guidance is provided for terminating the inspection process if all indications are that the building did not sustain damage requiring repairs. Where the building has sustained damage, the damage inspection reports and results of the structural analysis are subsequently used as input to the evaluation of earthquake damage to determine the need for repair or repair and retrofit, which is covered in Chapter 4.

3.2 General

As outlined in Figure 3-1, inspection and analysis include the following steps and the types of inspection introduced and defined in Section 2.2.3:

a) Preliminary inspection in accordance with Section 3.3.

b) Structural analysis (where required) of the building response to the damaging earthquake shaking in accordance with Section 3.4.

c) Identification of possible damage locations requiring detailed inspection in accordance with Section 3.5.

d) Detailed visual inspection in accordance with Section 3.6.

e) Reconciliation of results of structural analysis (as necessary) and reinspection (where required) in accordance with Section 3.7.

f) Intrusive inspection (where required) in accordance with Section 3.8.

The outcome of the inspection process is the determination of damage classifications of the structural components, which serves as the input for the damage evaluation of Chapter 4.

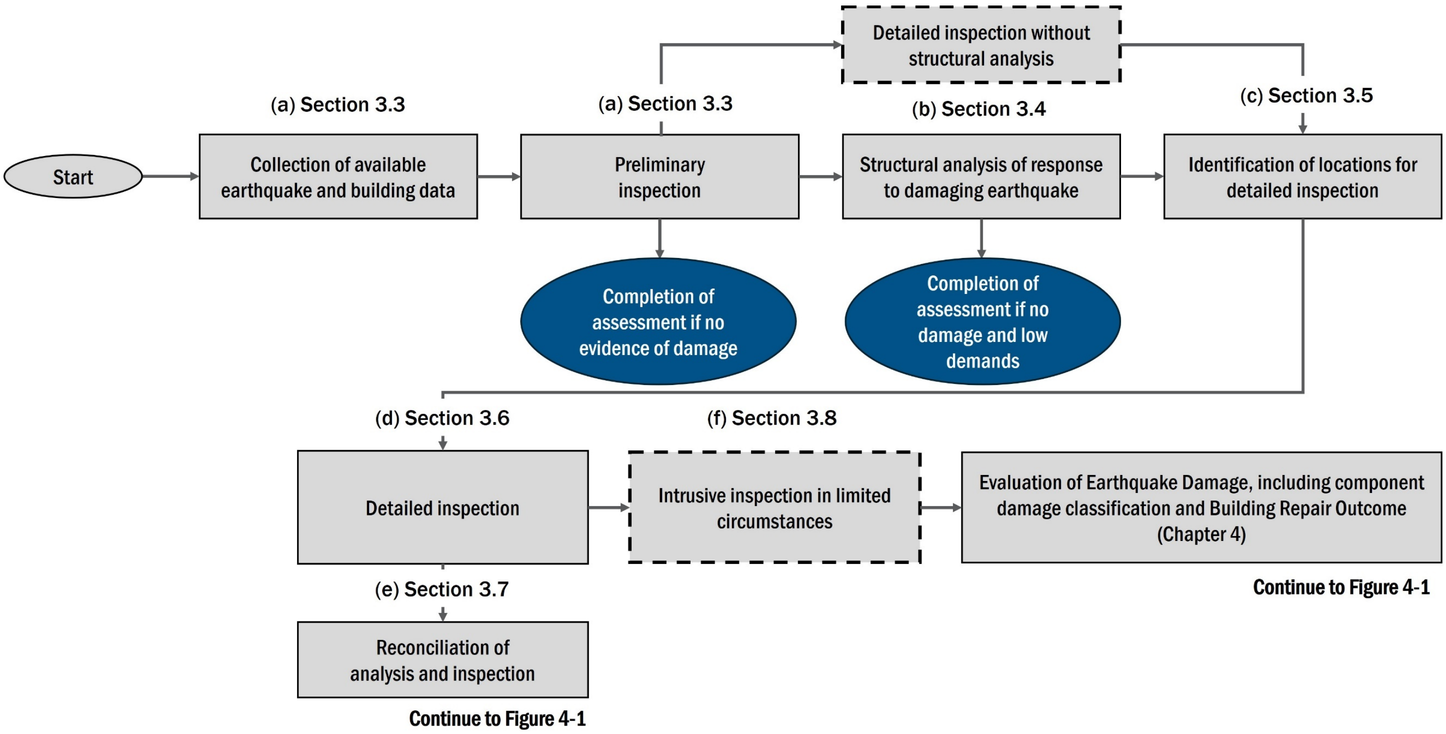


Figure 3-1 Overview of inspection and analysis process (adapted from Figure 2-2), identifying each step with references to relevant sections of Chapter 3. Dashed lines indicate steps that might not be taken in all assessments.

Exceptions:

It is permitted to forego steps (b) through (f) if the conditions of Section 3.3.5 are satisfied, that is, if the building can be deemed to be undamaged by virtue of the findings of the preliminary inspection (Section 3.3.5).

It is permitted to forego steps (c) through (f) if the conditions of Section 3.4.4 are satisfied, that is, if the building can be deemed to be undamaged by virtue of the findings of the preliminary inspection combined with the structural analysis.

It is permitted to forego the structural analyses in step (b) if the conditions of Section 3.3.6 are satisfied, and instead to proceed directly to the detailed inspection, in accordance with Section 3.5 and Section 3.6.

> *Commentary: The identification of earthquake damage is primarily based on a process of inspections to establish the damage classification of structural components. The process begins with a preliminary inspection to evaluate the overall building performance and to identify structural damage that is readily apparent (i.e., without destructive removal of architectural or other finishes to inspect structural components).*

In cases where the structure is fully exposed (e.g., open parking garage) or where damage is otherwise obvious (e.g., low-rise buildings with extensive damage), one can move from the preliminary inspection to detailed inspections (Section 3.5 and Section 3.6). Otherwise, these Guidelines *recommended procedures for structural analyses (Section 3.4) to help identify locations where damage is likely to have occurred, as inferred by the component forces and deformations induced by the earthquake ground shaking as estimated in the analyses.*

Detailed inspections (Section 3.6) generally entail removal of architectural or other finishes (e.g., thermal fire protection). The engineer is therefore faced with a tradeoff between the cost of detailed inspections and the confidence in identifying locations of structural damage. Section 3.5 provides recommendations on identifying locations for detailed inspections, beginning with those components that are most likely to have been damaged, as judged by preliminary inspections, knowledge of the expected building behavior, and structural component demands determined from structural analyses.

Where the locations of observed damage are inconsistent with the estimated demands from structural analysis, efforts should be made to reconcile the two (Section 3.7). Depending on whether damage is found at the expected locations, the number of detailed inspection locations can be adjusted.

Finally, in certain limited situations where visual inspection results cannot be reconciled with analyses and there is reason to suspect that there may be hidden damage, then intrusive inspections (Section 3.8) may be warranted, before proceeding to the evaluation phase (Chapter 4).

3.3 Preliminary Inspection

3.3.1 Earthquake Characteristics

Collect and analyze data on the earthquake ground shaking characteristics to develop a preliminary characterization of the ground motions that may have affected the building. Formulate an approximate response spectrum for the site suitable for preliminary estimation of response amplitudes for the building.

> ***Commentary:*** *These data will provide information about how strongly the earthquake shook the building, which is useful before conducting the on-site inspection. The first and most readily available estimate of earthquake ground motions are* PGA, PGV*, and spectral accelerations from ShakeMap (Worden et al., 2020), which combines information about the earthquake source, estimates from ground motion models, reported intensities, and recorded measurements of strong motions. The methods used by ShakeMap to combine these sources of data and interpolate between available information are well documented and vetted (Wald et al., 2021). However, while the ShakeMap ground motion values away*

from stations reflect the estimated site soil conditions on a regional scale, these may not accurately represent local site conditions that may affect the earthquake ground shaking. The best source of strong-motion data is from instruments (if any) that recorded ground shaking at the building location or immediately adjacent to the site. Alternatively, one may estimate ground shaking using recordings at nearby sites provided the site soil conditions, distance to the fault rupture, and other factors are similar. Finally, reports of damage to construction in the vicinity of the building may be useful in developing a preliminary estimate of ground shaking intensity.

3.3.2 Existing Building Data

Collect and review existing building data to develop an understanding of the building configuration, size, age, construction materials, structural framing system, nonstructural systems and finishes, prior structural modifications, and site conditions.

Commentary: Review of the existing building information serves several purposes. If reviewed before field investigations, the information facilitates the identification of structural components and helps to guide the field investigation to components that are likely to be damaged. Existing information can also help to distinguish between damage caused by the earthquake and pre-existing conditions. The following documents should be assembled, if available:

- *Construction drawings,*
- *Site seismicity/geotechnical reports,*
- *Structural calculations,*
- *Construction specifications,*
- *Contractors' shop drawings and other construction records,*
- *Foundation reports,*
- *Prior building assessments, including placarding (tagging) information,*
- *Street view images of the building, and*
- *Building instrumentation data (if available).*

Potential sources of construction drawings and other documents include the current and previous building owners, building departments, and the original architects or engineers. Drawings may also be available from architects or engineers who have performed prior evaluations for the building. In some cases, building recovery reports and pre-earthquake inspection data, gathered to facilitate rapid building assessment following an earthquake, may be available.

Not all the existing building data identified here needs to be reviewed prior to the site visit for preliminary inspection. Information not reviewed at this time should be considered for later phases of the inspection and analysis. Where available data are limited (e.g., there are no available plans), the engineer may need to document a greater amount of information in the site visit and detailed inspections about the as-built conditions.

3.3.3 Site Visit and Data Collection

Visit the site: to (1) perform a preliminary inspection for apparent damage to the structural and nonstructural components of the building, and (2) collect and confirm existing building data (Section 3.3.2).

The preliminary inspection should include visual observation of both the building interior and exterior. Selective, nondestructive removal of nonstructural finishes or coverings (e.g., ceiling tiles, access panels) facilitates visual identification of structural damage. More aggressive nondestructive intrusions may be warranted if hidden damage is suspected (e.g., the building appears out-of-plumb, observation of nonstructural damage, evidence of spalled concrete).

Refer to Chapter 5 for further guidance on characterization of Damage States for specific components and actions.

> *Commentary: A site visit is made to the building and a visual inspection is performed to identify obvious indications of structural or nonstructural damage. This inspection is independent of any inspections required to identify potential life-safety risks and placarding by the Authority Having Jurisdiction, although it may be informed by such prior inspections. Examples of structural damage include:*
>
> - *Cracking or spalling of concrete or masonry,*
> - *Yielding, buckling, or fracture of reinforcement in concrete or masonry structures or in connections and members of steel structures,*
> - *Splitting or fracture of wood components or connections,*
> - *Foundation settlement or tilting,*
> - *Residual (permanent) drift, and*
> - *Further examples are provided in Chapter 5.*
>
> *Determination of nonstructural damage is not the primary objective of these Guidelines. Observation of such damage, however, can serve as a proxy to infer likely locations of structural damage. Relevant nonstructural damage may include sliding or movement of joints, damage to gypsum-sheathed wall partitions, pounding (interaction) of secondary structural systems (e.g., stairs) with primary structural elements (e.g., walls or columns), damage to stairs, and broken glazing.*
>
> *The inspecting engineer should identify structural irregularities that can adversely affect performance, leading potentially to the concentration or amplification of damage. Such locations should be inspected for potential damage to the extent that they are visible during the preliminary inspection. Additionally, the inspecting engineer should evaluate whether observed damage is indicative of response that might be a result of a system irregularity that was not otherwise identified from the review of existing building data. Structural irregularities are defined by ASCE/SEI 41 and other sources and commonly include:*

- *Vertical irregularities, including discontinuous components of the seismic-force-resisting system, soft or weak stories, and foundations on slopes,*
- *Setbacks in the façade and floor plans,*
- *Horizontal (plan) irregularities, including open-front plans, or other torsionally unbalanced seismic-force-resisting systems, and*
- *Insufficient seismic separation between adjacent structures resulting in potential for pounding.*

The site visit should also be used to identify as-built conditions that differ from construction drawings and other available information, especially where construction detailing may have adversely affected the load path or led to the concentration of damage.

Observations from the site visit should be documented systematically, such that component damage can be mapped to the structural drawings and incorporated with the detailed inspection results (see Section 3.6).

Where significant damage is observed, restriction of access, and/or immediate temporary shoring may be required for the affected components (without further assessment). If an immediate safety condition is identified, the evaluating engineer has a professional obligation to inform the property owner and/or the building official of the condition, as appropriate. This type of damage and immediate actions should be addressed as part of the placarding (tagging) process by the Authority Having Jurisdiction and is not within the scope of these Guidelines*.*

3.3.4 Pre-existing Conditions

Evaluate observed damage to classify it as either pre-existing in the structure (before the damaging earthquake) or new (caused by the damaging earthquake). This classification should consider the expected behavior of the structure under earthquake shaking, as opposed to other types of loads or actions. Refer to Chapter 5 for guidance on the expected modes of seismic response for specific structural systems and materials, along with other types of pre-existing damage that may be encountered. Whether or not the damage is pre-existing, visual evidence of damage should be considered in the evaluation of the structure per Chapter 4.

Commentary: Evaluating the cause of damage requires an understanding of how the building is likely to have been affected by various loads, including self-weight, applied loads, prestressing, restraint of volume change, differential settlement, wind, and earthquakes. It is also useful to recognize the distinguishing features of older damage and more recent damage. The inspecting engineer should consider whether the observed damage correlates with the expected seismic response of the structure. If the observed damage is not reasonably consistent with the overall seismic behavior of the structure, the damage may have been caused by an action other than the earthquake. Pre-existing damage may also include damage from prior earthquakes, other hazards, or deterioration.

The presence of paint, glue, dirt, soot, or signs of corrosion in cracks typically indicates that the damage pre-existed the damaging earthquake. Similarly, identifying evidence of prior repairs (e.g., skim coat material, v-grooves at cracks, sealant at cracks) and assessing whether cracks appear old or fresh can help determine whether damage was caused by a recent earthquake.

3.3.5 Undamaged Structure

It is permitted to classify the building structure as an undamaged structure if there is no evidence to indicate that elements of the seismic-force-resisting system and gravity-force-resisting system have sustained more than minor earthquake damage (i.e., damage exceeding damage class DC0), as defined per Sections 2.2.1, 2.2.3, and 2.2.4.

A structure classified as an undamaged structure need not be further assessed for post-earthquake damage or repair.

Commentary: During the preliminary inspection phase, it is unlikely that the building structural system has experienced damage if there is no damage visible from inspecting readily accessible structural components where damage is likely to have occurred, and if one or more of the following criteria are met: (1) there is not more than slight damage to nonstructural finishes and components; (2) the earthquake ground motions are low (e.g., approximately less than 50% of the design earthquake intensity); (3) there are no reports of structural damage to nearby buildings of similar construction and age; and (4) the building satisfies benchmark building codes and standards for Life Safety Performance at BSE-1E in ASCE/SEI 41 (Table 3-2 in ASCE/SEI 41). In such cases, it is unlikely that additional study of the building will reveal damage that has not yet been identified, and it is acceptable to classify the building as undamaged and discontinue the post-earthquake assessment.

The absence of Performance-Critical Damage does not necessarily mean that the building will perform well in future earthquakes but, rather, only indicates that its future performance capability has not been reduced by the earthquake shaking that led to the inspection. If the preliminary inspection reveals information that indicates the building may have a high risk for future earthquake shaking, even in the absence of damage, then that information should be conveyed to the owner or other stakeholder(s) in a written report.

3.3.6 Detailed Inspection without Structural Analysis

Except as noted below, structural analysis is not required as part of the investigation to help identify locations of the structure requiring detailed inspection. Where detailed inspection is conducted without structural analysis, detailed inspection should be performed for all likely locations of structural damage in accordance with Section 3.5.

Instances where structural analysis should be used for the investigation of damage, unless visual inspection is performed for *all* structural components:

- Buildings taller than five stories,
- Buildings with structural irregularities, or
- Buildings whose seismic design would fall under the requirements of Risk Category III or IV.

Instances where structural analysis should be used for investigation of damage, without exception:

- Buildings for which analysis is needed to assess cyclic fatigue damage.

> Commentary: *In general, the damage identification should be based on detailed inspection of the structural components. Examples of cases where detailed inspections without structural analysis will generally suffice for the damage investigation include exposed structures where detailed inspections of all locations can be conducted with minimal effort (e.g., parking garage, metal building, one-story industrial/warehouse type buildings). In addition, even where the structure is not exposed, it may be more expedient to perform detailed inspections without structural analysis in low-rise buildings with regular geometries where drawings and information on the structural system are not available. The detailed inspection in these cases can follow the procedures of Section 3.5 and Section 3.6, including using the statistical sampling procedure to select/reduce locations to inspect (Section 3.6.2).*
>
> *Cases when structural analysis should be used are for: (1) taller buildings with large numbers of potential damage locations, where higher-mode effects may tend to trigger damage at various locations in the structure; (2) buildings with structural irregularities that may cause damage at locations that are not anticipated; and (3) buildings in higher risk categories (i.e., III or IV) that warrant higher assurances of identifying earthquake damage. Due to the complexity of these buildings and the inherent uncertainties in identifying damage locations, if structural analysis is not used in these cases, all structural components should be visually inspected in detailed inspection without any reduction from statistical sampling. Also, note that while structural analysis may not be required to guide inspections, structural analysis may still be required to evaluate the building for disproportionate earthquake damage or substantial structural damage.*
>
> *Other cases when structural analysis should be used for the damage identification include: (1) where detailed visual inspection may be insufficient to identify component damage that requires repair, such as cyclic fatigue damage to components; (2) instances where detailed inspection may be particularly difficult and costly and it is desired to minimize the number of detailed inspection locations (e.g., where removal of asbestos finishes or fire protection is required); and (3) where it is likely that structural analysis will be required for the damage evaluation in Chapter 4 (i.e., to evaluate whether the building is compliant with acceptable seismic codes or has experienced disproportionate earthquake damage or substantial structural damage).*

Finally, even if a comprehensive structural analysis (i.e., 3D computer model of full structure) is not performed, this guidance does not preclude the use of simpler methods of analysis to help characterize the overall building response or performance of individual components. For example, the engineer should avail themselves of (1) single-degree-of-freedom (SDOF) models or simplified formulae (e.g., target displacement formula from ASCE/SEI 41) to assess the earthquake displacement demands, and (2) idealized mechanism analysis of structural members to identify force-controlled versus deformation-controlled actions.

3.4 Structural Analysis of Building Response to Damaging Earthquake

3.4.1 Overview of Structural Analysis for Guiding Inspection

Choose a method of structural analysis to evaluate the structural response and component demands (forces and/or deformations) under a best estimate of the earthquake shaking that triggered the building inspection. Procedures for developing the structural model and applying the ground motion demands to the structural model should, in general, follow the methods of ASCE/SEI 41, but modified as necessary to reflect the following considerations:

- Accidental torsion provisions of ASCE/SEI 41 do not need to be considered provided that the analysis model reflects the known contributors to building torsion,
- Equivalent viscous elastic damping should be estimated according to requirements of ASCE/SEI 41 and may be adjusted to best represent the expected characteristics of the building site, foundation, structural components, and nonstructural components under the damaging earthquake, and
- Provisions for accounting for concurrent multi-direction effects may be adjusted depending on how the seismic ground motion demand is defined.

Commentary: *For post-earthquake assessment, in many cases, a linear procedure may provide a useful starting point, with a modal response spectrum analysis in accordance with the ASCE/SEI 41 linear dynamic procedure being the preferred option. If a ground motion recording is available at the building or a nearby site, a response history linear dynamic procedure may be an appropriate alternative. However, depending on the level of shaking and the structural system characteristics, the linear model may tend to overestimate the damage to some structural components. Prior studies (NIST, 2022) have indicated that both linear and nonlinear ASCE/SEI 41 models generally identify the story with the most damage and correctly identify component failure modes. Nonlinear models produce good estimates of drifts at critical locations. Linear models tend to underpredict drift demands and overpredict force and acceleration demands, especially in areas of the structure away from the critical damage.*

A variety of factors may motivate doing a nonlinear analysis. For larger or irregular buildings, and buildings with significant or extensive evidence of structural degradation or yielding, a nonlinear analysis may more efficiently guide inspections. Particularly in buildings where detailed inspection is difficult or expensive (e.g., due to removal of architectural finishes), it may be worthwhile to utilize nonlinear static or dynamic procedures to guide the inspection. When selecting the analysis procedure, the inspecting engineer should also consider other analysis needs in these Guidelines*. If a nonlinear model will be developed for other purposes (see Section 2.1.7), it may not increase the level of effort to use that same model here.*

Whereas ASCE/SEI 41 procedures are geared toward ensuring building safety, the analyses used to guide inspection are geared toward representing the building as realistically as possible. Therefore, requirements of ASCE/SEI 41 associated with accidental torsion, viscous damping, and multi-directional earthquake effects need not be followed if they influence aspects of structural response that are not likely to have affected the building response during the damaging earthquake.

3.4.2 Structural Analysis Model

Develop a structural analysis model of all structural and foundation elements that can be expected to substantially affect dynamic response of the building. To model the seismic response as realistically as possible, the analysis model should be based on expected values of material properties, component stiffnesses. component strengths (for nonlinear models), masses, and gravity loads.

It is acceptable to represent gravity loads, Q_G, using a single load combination accounting for dead, live, and effective snow loads:

$$Q_G = Q_D + Q_L + Q_s \tag{3-1}$$

where:

Q_D = action caused by dead loads

Q_L = action caused by the live load present at the time of the earthquake

Q_S = action caused by estimated snow load present at time of the earthquake

Equation 3-1 is consistent with Equation 7-3 of ASCE/SEI 41. The evaluating engineer should account for any significant discrepancies in the observed gravity loads and their distribution at the time of the earthquake from those documented on the construction drawings.

For dynamic analysis, seismic mass should be estimated based on the requirements of ASCE/SEI 41. The evaluating engineer should account for any significant discrepancies in the observed masses and the distribution at the time of the earthquake from those documented on the construction drawings.

Commentary: The live load can be estimated as 25% of the unreduced design live load if other information is not available.

For a symmetric or near-symmetric moment frame building, two-dimensional models of the structure in each orthogonal direction may be sufficient. However, for linear analysis there may be little time savings or other practical benefits associated with using two-dimensional models. Three-dimensional models are therefore expected to be used in most cases and are essential in cases where the torsional behavior may strongly influence response.

While a structural analysis model intended for seismic design of a new building may omit some components (e.g., gravity system components), structural analysis models used to interrogate response during the damaging earthquake should include all components of the building that can significantly influence the structural response under earthquake ground motions. This should include both structural and any participating nonstructural components (e.g., substantial architectural walls or cladding, such as masonry infills) of the superstructure. In many cases, a fixed-base structural model may be a reasonable starting point. Where foundation damage exists, or there is evidence of settlement, and if inspections and analysis show poor agreement, the foundation and soil models should be refined (e.g., using ASCE/SEI 41 Chapter 8).

It is outside of the scope of these Guidelines *to provide guidance on developing nonlinear simulation models. ASCE/SEI 41 catalogs nonlinear modeling procedures and modeling parameters for structural components.*

3.4.3 Seismic Demand Representation

Develop a representation of the ground shaking demand in the damaging earthquake using available nearby ground motion recordings and ShakeMap estimates of ground shaking intensity measure(s) for the event of interest. Depending on the type of structural analysis procedure, the earthquake ground motion may be characterized by either: (1) spectral accelerations at periods corresponding to one or more modes of vibration of the building, or (2) one or more sets of recorded ground motion histories. Procedures for applying the ground motion demands to the structural model should, in general, follow the methods of ASCE/SEI 41, but with the following modifications:

- Where the ground motion input of the damaging earthquake is represented by two unique sets of orthogonal spectral accelerations for use with the modal Linear Dynamic Procedure (LDP), then concurrent multi-direction effects in the X-Y directions may be combined using modal combination rules (e.g., Wilson et al., 1995; Menun and Der Kiureghian, 1998), rather than the 100%-30% method specified in ASCE/SEI 41.

- Where the ground motion input of the damaging earthquake is represented by one or more pairs of ground motions for the LDP or Nonlinear Dynamic Procedure (NDP), the orthogonal horizontal components of each ground motion should be rotated to the building's orientation and applied concurrently to the structural analysis model. Where vertical dynamic response of the structure is significant, the vertical mass should be incorporated in the analysis model and the vertical ground motion effects can either be considered by combining the results from a vertical response spectrum analysis with those of the analyses of the horizontal ground motions or by applying the vertical component of ground motion concurrently with the lateral components.

- Where more than one set of recorded ground motions or ground motion spectra are used to characterize the damaging earthquake, the structural analysis should be conducted for each ground motion set, and the resulting structural demands should be combined and evaluated in post-processing, as described Section 3.4.4.

Commentary: Recommendations for determination of the seismic demand representation depend on the available information. Where earthquake ground motions are recorded at or near the building site, these can be used directly for linear or nonlinear response history analysis of a building. It is recommended that representative ground motions be selected as close as possible to the site, generally within 1 km from a recording site, with similar subsoil conditions (i.e., same site classification) and distance to the earthquake rupture. ShakeMap provides a comparison of spectra of nearby recordings with the spectra estimated by ShakeMap at the site. There can be variations in shaking intensity and frequency content even at relatively close distances, e.g., as high as up to two-times in the short-period region (less than 1 second) within a half kilometer distance (Wald et al., 2021).

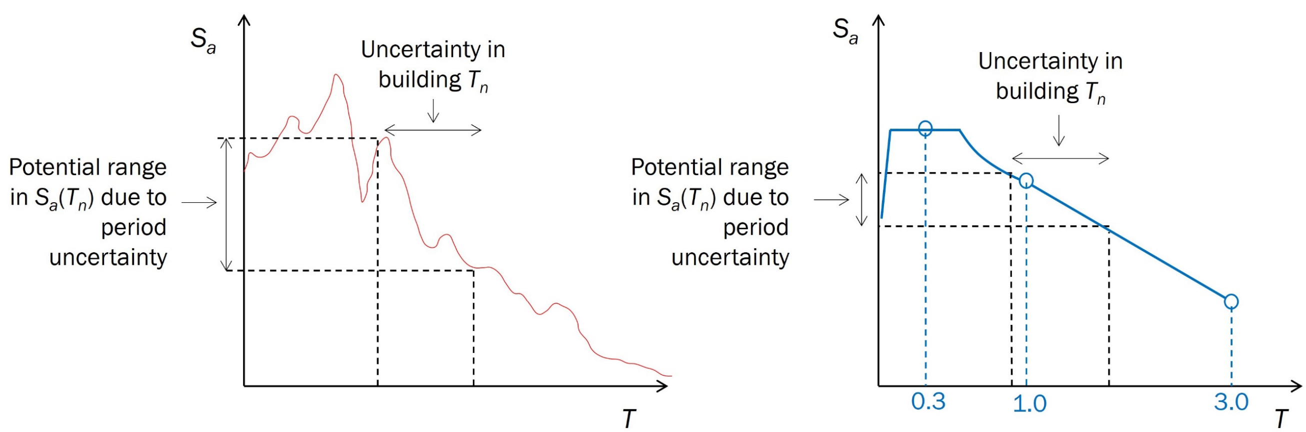
Where multiple nearby ground motions are available on similar ground conditions to that of the building in question, it may be useful to perform separate analyses for each of the ground motions and then evaluate the performance considering the distribution of response quantities from all records.

Recorded ground motions at or near a building site can also be used to create smoothed response spectra for use in modal response spectrum analysis of a linear model of the building. The response spectra may overestimate peaks and valleys and results may be sensitive to the estimated building period (Figure 3-2). Therefore, some smoothing procedure is generally recommended. One possible smoothing approach is provided in Chapter 3 of NIST GCR 22-917-50, Benchmarking Evaluation Methodologies for Existing Reinforced Concrete Buildings *(NIST, 2022).*

*Where recordings are not available, the site-specific shaking and V_s30 value used in ShakeMap can be determined with the online ShakeMap Sampling Tool (SST; Thompson et al., 2024). ShakeMap shaking intensity measures (*PGA, PGV, *and* $S_a(T)$ *at multiple periods) from the damaging earthquake can be used with the modal response spectrum analysis. ShakeMap values contain inherent smoothing due to the use of ground motion models, spatial interpolation of subsoil conditions, and statistical averaging with spectra from recorded motions (Wald et al., 2021). Although ShakeMap in the past has presented only* $S_a(T)$ *values at* T = *0.3s, 1.0s, and 3.0s, ShakeMap for future damaging domestic earthquakes will provide 22 combinations of* $S_a(T)$ *and period (Thompson et al., 2024), as in the USGS National Seismic Hazard Maps. In addition, ShakeMap reports the peak (maximum* *of two arbitrarily oriented horizontal components) response quantities, which would be a conservative estimate of simultaneous ground shaking in two horizontal directions (Worden et al., 2020). So, for three-dimensional response spectrum analyses, one may define two ground motion spectra (for each horizontal direction of the building) from the single ShakeMap spectra, using expected ratios between maximum, geomean and minimum direction spectra (Boore and Kishida, 2017). Note that ShakeMap peak values are not the maximum component considering all possible orientations of the components, i.e., it is not RotD100.*

Where response spectrum analysis is used, the guidance on Concurrent Seismic Effects in ASCE/SEI 41 Section 7.2.5.1 can be adopted. When linear modal response spectra or modal response history analyses are run based on a pair of spectra that represent multi-directional (orthogonal) earthquake motions, for the purposes of estimating response to guide post-earthquake inspections, it is recommended to use the SRSS (or alternate CQC3) method to combine the orthogonal effects, rather than the 100%-30% procedure that is specified in ASCE/SEI 41 (Wilson et al., 1995; Menun and Der Kiureghian, 1998).

Where recordings are not available at the building site, the ground shaking representations will not capture site-specific soil conditions and associated amplifications nor effects of soil-structure interaction. In addition, none of the ground shaking representations account for settlement and other forms of ground movement that may influence demands. These limitations may affect the interpretation of the analysis and reconciliation of analysis and inspection. Where site specific effects are critical to assessing the building damage, a more detailed geotechnical investigation and site-specific hazard analysis may be warranted.



(a) **(b)**

Figure 3-2 Illustration of response spectra generated from: (a) nearby ground motion recording and (b) ShakeMap estimates.

3.4.4 Critical Earthquake Demands

Use the structural analysis model (Section 3.4.2) and seismic demand representation (Section 3.4.3) to estimate: (1) peak story drift demands; (2) peak component deformation demands; and (3) peak component force demands.

Commentary: When more than one set of nearby ground motion recordings are used for the input to the structural analysis model, it is recommended that the peak responses (i.e., story drifts, component deformations, and component forces) initially be estimated using the average (mean) peak demand parameter, calculated from all the considered ground motion sets. To improve the reconciliation of analysis and inspection results per Section 3.7, the engineer may wish to consider calculating a weighted average or envelope of demands, calculated based on the individual ground motion records. Additionally, ground motions that produce responses that are significantly inconsistent with the observed performance may also be removed or replaced with analyses using another set of nearby records (see Section 3.7 for more guidance on reconciliation of inspection and structural analysis).

3.4.5 Undamaged Structure

It is permitted to classify the building structure as undamaged if the following criteria are satisfied:

a) The preliminary inspection of Section 3.3 has not identified damage from the damaging earthquake in elements of the seismic-force-resisting system and gravity-force-resisting system exceeding DC0, and

b) The critical earthquake demands in components of the seismic-force-resisting system and gravity-force-resisting system, determined in accordance with Section 3.4.3, do not exceed DC1. Refer to Section 5.4 for information on the Performance-Critical Damage limits.

A structure classified as an undamaged structure need not be further assessed for post-earthquake damage or repair.

Commentary: At the end of the structural analysis phase, if conditions (a) and (b) are satisfied, it is unlikely that additional study of the building will reveal damage that has not yet been identified. In such cases, it is acceptable to classify the building as undamaged and discontinue the post-earthquake assessment. Findings should be documented in a written report. Note that the building owner may still opt for some repair targeting objectives other than safety (e.g., cosmetic, durability, or serviceability).

A building structure whose components satisfy Section 3.3.5 but that has experienced moderate to significant damage to nonstructural components or experienced strong ground shaking is an example of a building whose structural system could be classified as an undamaged structure if it meets the requirements of Section 3.4.5.

The absence of Performance-Critical Damage does not necessarily mean that the building will perform well in future earthquakes but, rather, only indicates that its future performance *capability has not been reduced by the previous earthquake shaking. If the preliminary inspection and structural analysis reveal information that indicates the building may have a high risk for future earthquake shaking, then that information should be conveyed to the owner or other stakeholder(s) in a written report.*

3.5 Identification of Possible Damage Locations

3.5.1 General

Identify possible damage locations for detailed inspection (referred to as *inspection locations*) based on the following:

- Preliminary inspection in accordance with Section 3.3.3,
- Conditions always requiring inspection in accordance with Section 3.5.2,
- Identification of structural irregularities in accordance with Section 3.5.3,
- A mechanism analysis in accordance with Section 3.5.4, and
- Earthquake demands calculated from structural analyses in accordance with Section 3.5.5.

Subject to the exceptions in Section 3.3.6, it is not necessary to conduct structural analyses (Section 3.5.5) and to instead use visual inspection to identify locations for detailed structural inspection. Once the locations of detailed inspection are established, the number of inspection locations may be reduced, based on the guidance in Section 3.6.2.

Commentary: The method(s) used to determine the locations for detailed inspection can vary significantly from building to building. As described in Section 3.3.6 and illustrated in Figure 3-1, where the structure is fully exposed (not concealed behind architectural or other enclosures), one can use preliminary visual inspections in lieu of a comprehensive structural analysis (Section 3.5.5) to identify locations for detailed structural inspection. When this "without structural analysis" option is chosen, the requirements and procedures of Sections 3.5.2, 3.5.3, and 3.5.4 still apply.

3.5.2 Conditions Always Requiring Inspection

Identify conditions always requiring inspection. Section 5.3.4 defines specific conditions that always require inspection, regardless of the outcome per Section 3.5.3 through Section 3.5.5. These conditions do not apply if the structure is found to be undamaged as per Section 3.3.5 or Section 3.4.5.

If there are relatively few components with conditions requiring inspections (e.g., fewer than 20 components), then inspecting all of them is advisable; however, if there are more, then the detailed inspection locations can be adjusted as described in Section 3.6.2.

Commentary: Chapter 5 identifies specific components and conditions that are particularly vulnerable to damage and subject to rapid degradation in load-carrying capacity once DS2 is reached and potentially a high consequence of failure. These conditions are typically due to a component's non-ductile or force-controlled behavior and are often accompanied with a high level of uncertainty in expected performance. Due to this uncertainty in performance and potential consequence of failure, it is recommended that such components are always inspected. Examples for reinforced concrete structures (see Section 5.3.4) include non-ductile columns or walls with high axial load ratios, slender walls, and heavily loaded slab-column connections.

3.5.3 Inspection Locations Based on Mechanism Analysis

Using available information about the structural system, perform a mechanism analysis to identify locations for detailed inspection based on likely locations of inelastic actions in the structure. The mechanism analysis may be either qualitative to simply identify likely yielding locations, or quantitative to both identify likely yielding locations and to rule out locations that are unlikely to yield or be damaged. Additionally, the quantitative mechanism analysis can be used to limit or bound the component demands obtained from the structural analysis of Section 3.5.5.

Commentary: In consideration of possible inaccuracies in ground motion estimation and structural analysis, inspection locations identified based on calculated earthquake demands (Section 3.5.5) may either miss or overpredict potential locations of damage. Moreover, even when identifying detailed inspection locations based on a preliminary visual investigation of an exposed structure, it is useful to have an expectation of the structural response and locations where damage is most likely. A mechanism analysis may help understanding of likely building behavior and, thereby, (1) avoid missing damage or (2) limit the necessary inspection locations (e.g., by determining the capacity-limited axial force that can be imposed on a shear wall by yielding coupling beams or the axial force on a column by yielding buckling restrained braces or link beams in eccentrically braced frames). Depending on the circumstances and available information (e.g., buildings where detailed structural drawings may or may not be available), the mechanism analysis can vary in rigor. Where detailed information on the structural system and components is not known, an approximate (qualitative) mechanism analysis can be used to identify locations where damage is most likely. Alternatively, where detailed structural information is known, the mechanism analysis can be a Nonlinear Static Procedure analysis per ASCE/SEI 41, a plastic mechanism analysis per FEMA P-2018 (FEMA, 2018a), or alternative procedures that identify the primary yielding mechanism(s) of the structure under lateral loading.

3.5.4 Inspection Locations Based on Structural Irregularities

Study the structure to identify likely locations of damage based on structural irregularities, which may also warrant Detailed Inspection.

> *Commentary: A study of the building configuration can help to identify structural configuration irregularities that are known to be associated with building damage, to improve understanding of building behavior, and to avoid missing damage. The structural configuration irregularities could be identified from ASCE/SEI 41 or other sources or from project-specific guidance. The study need not include additional numerical structural analysis, but instead can be a review of the structural configuration to qualitatively identify additional inspection locations based on identified irregularities.*

3.5.5 Inspection Locations Based on Calculated Earthquake Demands

When structural analyses are performed to calculate the earthquake demands in accordance with Section 3.4.3, determine locations requiring detailed inspections (Section 3.6) based on the requirements of this section for deformation-controlled and force-controlled actions, in addition to the requirements of Sections 3.5.2, 3.5.3, and 3.5.4.

Deformation-Controlled Actions: All locations where calculated earthquake demands from Section 3.4.3 exceed the *Inspection Indicator* for deformation-controlled actions, defined per (1) or (2) below, shall be inspected in the detailed inspection (Section 3.6).

1. Where the component demands are determined using a nonlinear structural analysis, the Inspection Indicator, I_p or I_t, shall be given by Equation 3-2a for components where modeling parameter *a* is specified in ASCE/SEI 41, or Equation 3-2b for components where modeling parameter *d* is specified in ASCE/SEI 41.

$$I_p = C_i \ \eta \ a \tag{3-2a}$$

$$I_t = C_i \ \eta \ d \tag{3-2b}$$

where:

C_i = the inspection factor specified in Table 3-1

η = a multiplier specified in Section 5.4 to adjust between ASCE/SEI 41 modeling parameters (*a* or *d*) and estimate of deformation at DS2

I_p shall be compared with plastic deformation (rotation, displacement) demands from Section 3.4.4, while I_t shall be compared with total deformation (rotation, displacement) demands from Section 3.4.4.

2. Where the component demands are determined using a linear structural analysis, the Inspection Indicator, I, shall be given in terms of Equation 3-3a for components where modeling parameter *a* is specified in ASCE/SEI 41, or Equation 3-3b for components where modeling parameter *d* is specified in ASCE/SEI 41.

$$I = C_i \ \eta (m/0.75) \tag{3-3a}$$

$$I = C_i\,[\,\eta(m/0.75) + 1 - \eta\,] \tag{3-3b}$$

where:

C_i = the inspection factor specified in Table 3-1

η = a multiplier specified in Section 5.4 to adjust between ASCE/SEI 41 modeling parameters (*a* or *d*) and estimate of deformation at DS2

Unless otherwise specified in the material chapters, *m* is the component capacity modification factor (*m*-factor) from ASCE/SEI 41 for the primary system collapse prevention (CP) limit. The Inspection Indicator, I, shall be compared with the demand-capacity-ratio obtained from the analyses in Section 3.4.3, consistent with linear procedures in ASCE/SEI 41.

Force-Controlled Actions: All locations where calculated earthquake demands, calculated by either a linear or nonlinear analysis, from Section 3.4.4 exceed the Inspection Indicator for force-controlled actions, defined per the following equation, shall be inspected in the detailed inspection (Section 3.6):

$$I = C_i\,Q_{ce} \tag{3-4}$$

where:

C_i = the inspection factor specified in Table 3-1

Q_{ce} = the expected strength of the component, defined per ASCE/SEI 41 and modified, as applicable, per Chapter 5.

As described in Table 3-1, the inspection factor shall be chosen based on the uncertainty in the calculated earthquake demands, considering uncertainties in the seismic demand input, the structural analysis model, and knowledge of the defining parameters and of the behavior of the structural components.

Table 3-1 Inspection Factors Based on Uncertainty in Building Response

Uncertainty	Inspection factor, C_i	Description
Low	0.6	Ground motion instrument available at the building site; analysis model is validated and well developed; component failure modes are well understood
Medium	0.5	More uncertainty in one of the above criteria – for example, ground motion instrumentation on sites within 5 km on the same site class; either analysis model or component failure mode is more uncertain than the criteria for "low uncertainty"
High	0.4	Limited or no nearby ground motion instrumentation; significant uncertainties in structural analysis model or component failure mode

Commentary: *Chapter 5 defines the Performance-Critical Limits for components that correspond to DS2, which defines the lower limit of the DC2 Damage Class. As described in Chapter 5, the Performance-Critical Limits for deformation-controlled components are obtained by adjusting the* a *and* d *modeling parameters of ASCE/SEI 41 to reflect the point of initiation of component strength loss. For reinforced concrete, the default adjustment factor, η, is 0.75, i.e., where the Performance-Critical Limit is 0.75 times the* a *or* d *modeling parameter (see Section 5.4 for more details and exceptions to the default value).*

For establishing the Inspection Indicator, the Performance-Critical Limits are reduced by the inspection factor, C_i, in Table 3-1 so as to reduce the likelihood of missing critical damage during detailed inspection. The inspection factor depends on the uncertainty in both the calculated demands (from analysis) and in the Performance-Critical Limits. The derivation of the Inspection Indicators is described in Appendix A.

As outlined in Table 3-1, establishing the uncertainty is left to the judgment of the engineer, considering: (1) the availability and proximity to measured ground motions to define the seismic demands; (2) whether drawings and other information are available to establish the as-built properties of the structure; and (3) how well the structural analysis model can simulate the expected behavior. The latter point depends on both the characteristics of the structural analysis (e.g., linear versus nonlinear analysis, static versus dynamic analysis, uniaxial plastic hinge versus more detailed fiber or multi-axial models) and confidence in understanding of the structural behavior (e.g., well-controlled yielding of code-conforming components versus non-conforming components where multiple failure modes with rapid onset of degradation are possible). The inspection factors in Table 3-1 (from 0.4 to 0.6) target a less than roughly 10% probability that the component that is not inspected has experienced component strength loss during the damaging earthquake (See Appendix A).

Where linear analysis is used, Performance-Critical Limits and inspection indicators are defined in terms of m-factors from ASCE/SEI 41. Equations 3-3a and 3-3b adjust the

ASCE/SEI 41 m-*factors to represent DS2 (initiation of component strength loss). Furthermore, the 0.75 factor in Equations 3-3a and 3-3b is intended to back out the additional conservatism built into the elastic analysis procedures and* m-*factors in ASCE/SEI 41. The resulting Performance-Critical Limits are multiplied by the same inspection factors as applied to the component deformation limits.*

Figure 3-3 shows two examples comparing the earthquake demand-capacity ratios (DCRs), calculated by linear modal response spectrum analyses, to the observed component damage, classified by Damage Class. One example (Figure 3-3a) is a coupled concrete shear wall building, which was damaged in the 2011 Canterbury, New Zealand Earthquake, and the second (Figure 3-3b) is a concrete moment frame, which was damaged in the 1994 Northridge, California Earthquake. The shading on the shear wall elevation is coordinated based on the Performance-Critical Limits (termed "DS2" in the figure) and the Inspection Indicator. Red shaded wall segments have DCRs that exceed the Performance-Critical Limit, and orange shaded segments have DCRs that exceed the Inspection Indicator using Equation 3-2a, thus defining the locations for detailed inspections. The green shaded wall segments are below the Inspection Indicator and do not require inspection. The Inspection Indicator is based on an assumed inspection factor of 0.5, reflecting a medium level of uncertainty (Table 3-1) given the availability of representative ground motions, detailed structural design drawings for the building, and use of a detailed three-dimensional structural analysis model. Similarly, the frame elevation in Figure 3-3b overlays the calculated DCRs, Inspection Indicator status (orange and red squares require inspection), and the observed Damage Class. In both examples, the locations that are identified as requiring detailed inspections (shaded orange and red) significantly exceed the observed number of locations where the observed visual damage indicated that there was Performance-Critical Damage. This is intentional to ensure there is a low probability of missing critical damage during the detailed inspection. As described in Section 3.6.2, the process for performing detailed visual inspections allows for reducing the inspection locations, depending on whether damage is observed, beginning with observations at locations with the highest DCRs.

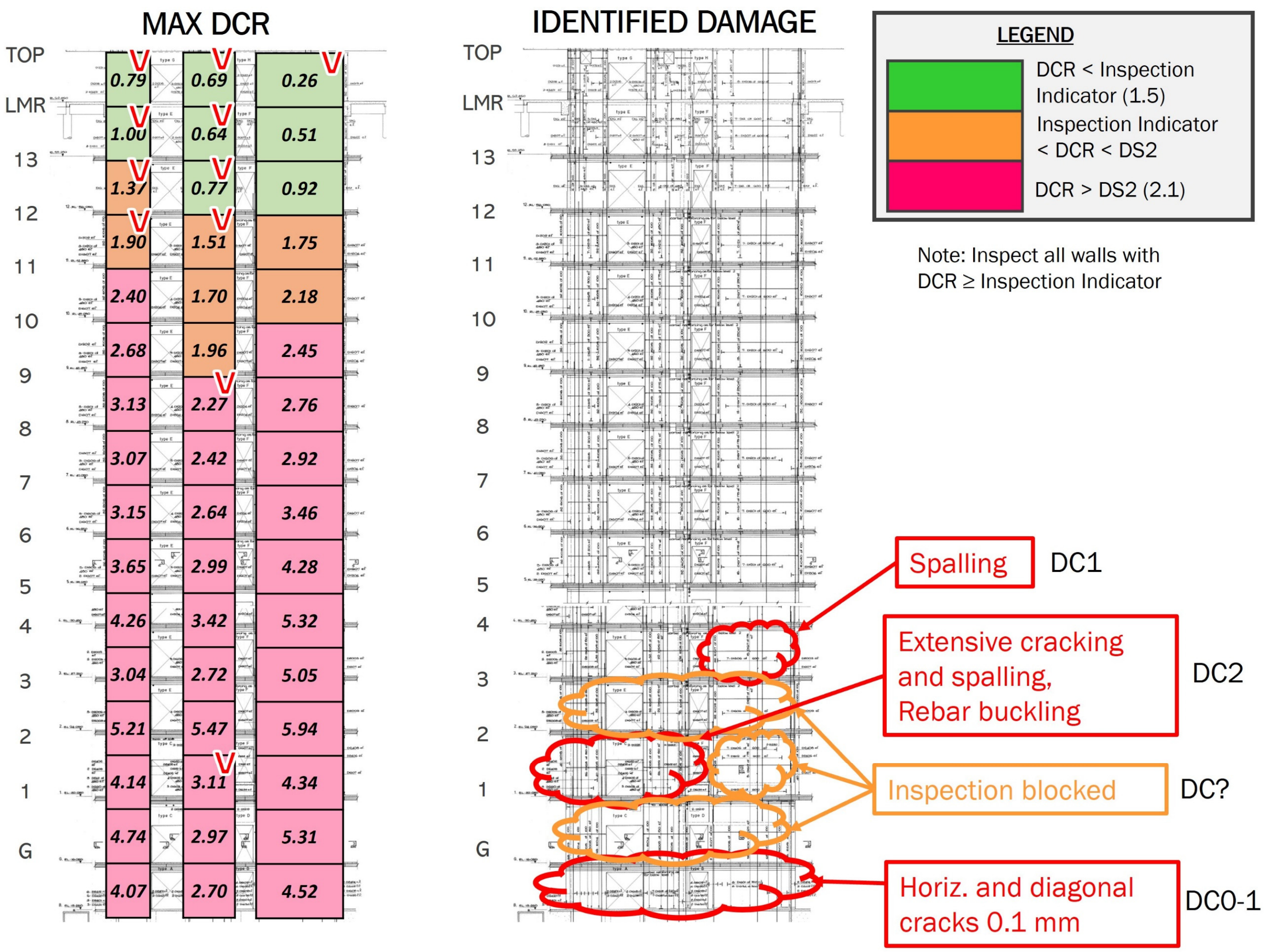


Figure 3-3a **Example of damage classification from Visual Damage States compared to inspection locations for concrete shear wall. A red "V" in the left-hand figure indicates that the wall segment is shear controlled.**

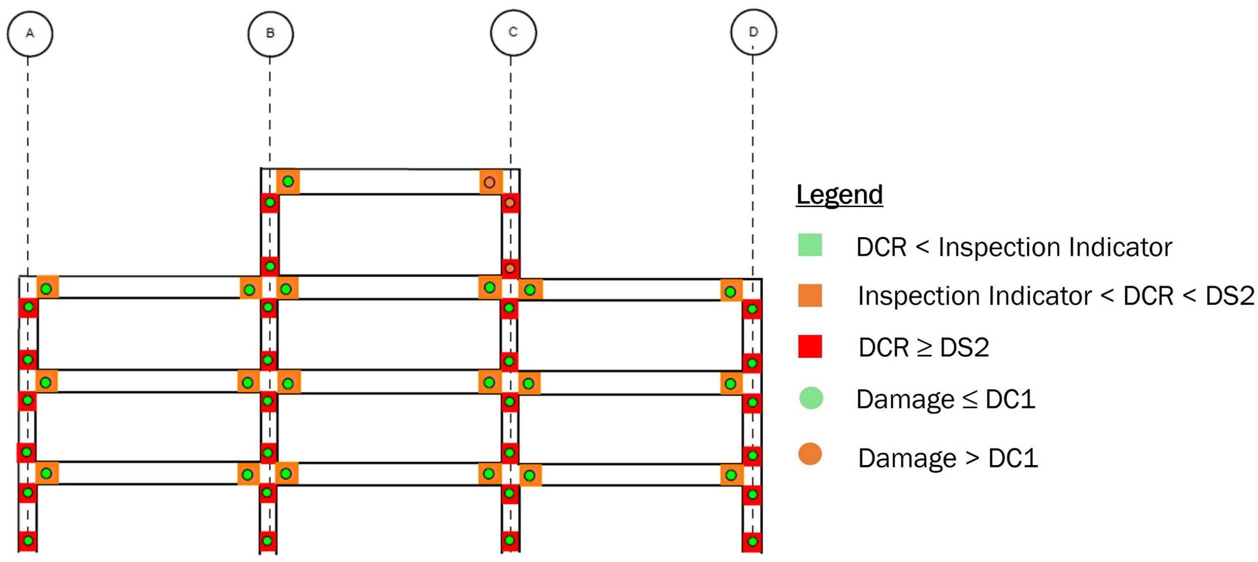


Figure 3-3b Example of damage classification from Visual Damage States compared to inspection locations for concrete moment frame. In this example, all frames require inspection (i.e., all are colored orange or red).

3.6 Detailed Visual Inspection

3.6.1 Detailed Visual Inspection

Conduct a detailed visual inspection of the possible damage locations indicated in Section 3.5. At each visual inspection location, the surface of the structural component shall be exposed such that surface damage on the structural component can be visually observed.

The visual inspections shall document the Damage Class for each component (defined in Chapter 2), consistent with the Visual Damage States for each component per Section 5.5.

> ***Commentary:*** *Visual observation of the surface of a structural component may vary depending on the type of structural component or material. See Sections 5.3 and Section 5.5 for material-specific inspection techniques and Visual Damage States.*
>
> *Removal of nonstructural finishes including ceilings, gypsum board enclosures, and fireproofing should be carried out where required to enable access to and visual observation of the surface of the structural component (e.g., observation of cover concrete or welds for cracking, structural steel surfaces for local or global buckling).*

3.6.2 Reductions to Detailed Visual Inspections

It is permitted to reduce the number and locations of detailed visual inspections (Inspection Locations, IL) to fewer than those that are identified by Section 3.5 through judgment supported by prior knowledge of the expected structural system response, trends observed in the structural analysis and inspections, and statistical sampling. Any reductions in the number and location of detailed visual inspections shall be clearly documented and justified in the written report of the inspection process and findings.

> *Commentary:* *Since the threshold for detailed visual inspection, as outlined in Section 3.5, is intentionally conservative, it is likely that some of the locations identified as requiring detailed visual inspection will be found to be undamaged. For this reason, it is permitted to reduce the number and locations of detailed visual inspections when there is strong evidence to suggest that further inspections are not necessary. In this regard, the damage inspection is an evolving process, informed by knowledge of the structural system, patterns of damage observations to structural and nonstructural components, and potential consequence of undetected damage.*
>
> *The following steps are intended as guidelines to apply in conducting detailed visual Inspections:*
>
> 1. *Detailed visual inspections should generally begin in locations, i.e., critical regions, where damage is most likely to have occurred, based on the procedures described in Section 3.5. Where structural analyses have been performed, the likely damage locations would typically be at the floor(s) with the largest story drift demands and the highest calculated ratios of earthquake demands to the component Performance-Critical Limits. Where structural analyses are not performed, floors with the largest story drift demands and highest earthquake demands can be inferred from mechanism analyses (Section 3.5.3), locations of irregularities (Section 3.5.4), and damage to nonstructural architectural enclosures, partition walls, and façade walls. When inferring structural damage from observations of nonstructural components, one should consider the flexibility of the nonstructural component and its attachment to the structure. In cases where brittle architectural finishes are rigidly attached to the structure (e.g., plaster on lath that is fit tight to the structure), damage to the finishes is likely to reflect the deformation demands on the underlying structural components. On the other hand, the absence of damage to finishes with flexible attachments (e.g., partial height gypsum board partitions, connected to the structure with flexibly cold-formed steel studs) does not necessarily imply low demands on the underlying structure.*
> 2. *Once detailed inspections are underway, detection of damage or lack of damage can be used to refine the locations for detailed inspections. For example, as suggested by the method in Figure 3-4, the inspection process should start by requiring inspection of 100% of the critical regions identified in step 1. Initially, a minimum of 5% of the identified damage locations should be inspected, and if all the consecutive damage locations for the component type are identified as undamaged (DC0), then the inspection frequency may be progressively reduced in that region of the structure. If damage is identified in subsequent inspections, then the inspection should revert to 100% of the locations within the affected portion of the structure.*

3. *The process outlined in steps 1 and 2 should be repeated in other regions of the structure with the next highest likelihood of damage. If these subsequent inspections identify the regions as undamaged, then it may be possible to further reduce the number of inspections in other areas of the building structure. For example, if inspections of several stories of the structure with decreasing likelihood of damage consistently identify components as undamaged, then it may be permitted to conclude the detailed visual inspections. In addition to differentiating the damage regions by story level, other features of the structural system to consider in determining inspection progression include: (1) framing lines; (2) vertical stacking of components (e.g., link beams in coupled shear walls or eccentrically braced frames); (3) irregularities (Section 3.5.3); and mechanisms (Section 3.5.4).*

Ultimately, the engineer of record for the inspections is responsible for determining the required extent of the detailed visual inspections.

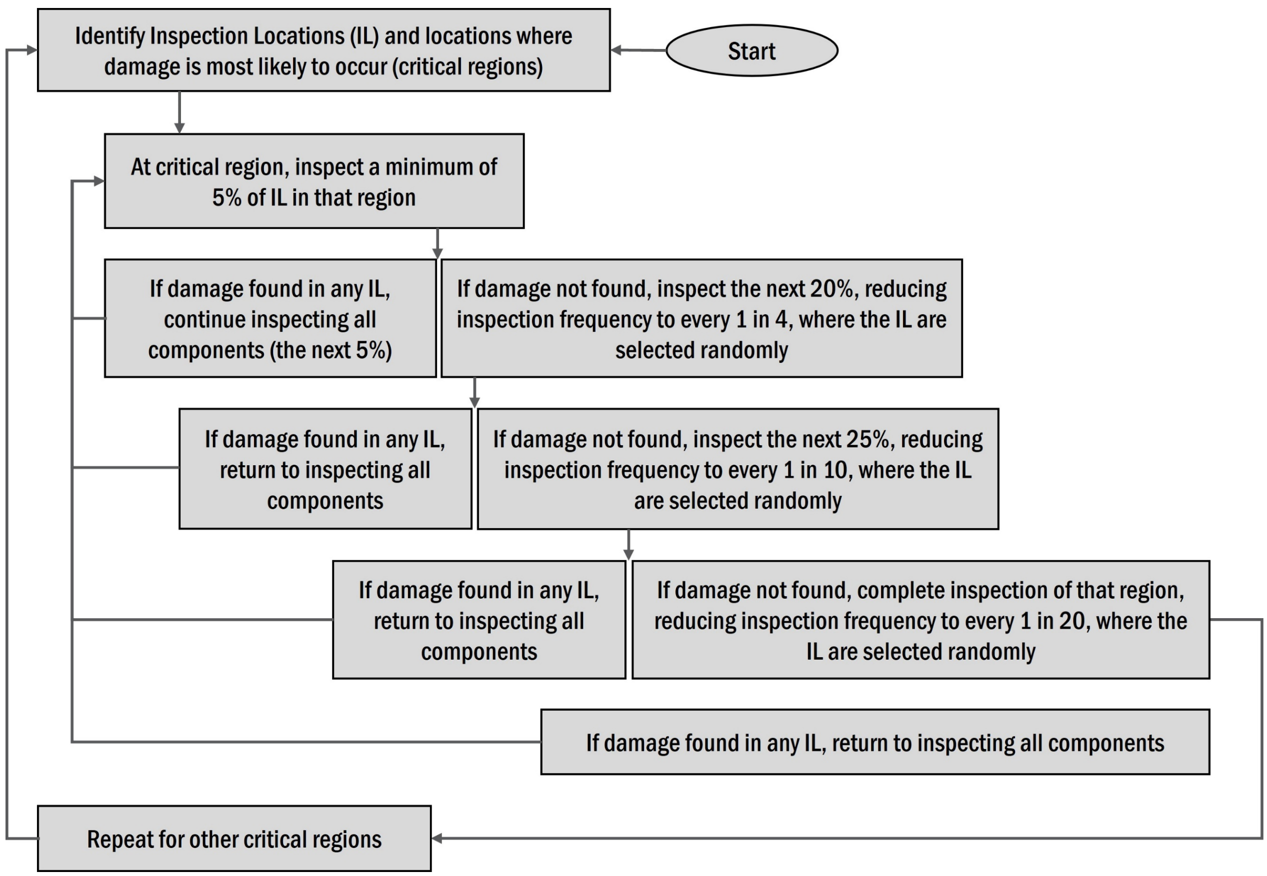


Figure 3-4 Statistical sampling process for establishing Inspection Locations (IL) for detailed visual inspections (adapted from AS/NZS 1554.1, 2004).

3.7 Reconciliation of Inspection and Structural Analysis

3.7.1 Comparison of Analysis with Observed Damage

When structural analyses are conducted to guide the inspection, the predicted damage locations based on demands calculated by structural analysis (Section 3.5) shall be compared with the damage observed in the detailed visual inspection (Section 3.6). Where the calculated and observed results are not in acceptable agreement, in terms of the severity, distribution, or type (mode) of damage, the structural analysis shall be reconsidered to determine whether an improved comparison can be obtained through reasonable adjustments in the ground motion estimation and the structural modeling approach. Particular attention shall be made to reconciling conditions that always require inspection (per Section 3.5.5), where the analysis estimates Performance-Critical Damage (i.e., damage class of DC2) and the visual inspection classifies the damage class as less than DC2.

> Commentary: *Discrepancies between observed and calculated results may be indicative of a lack of clarity on the likely performance of the building during the damaging earthquake. The purpose of this step in the inspection and analysis process is to ensure the performance and damage to the building in the damaging earthquake are well understood. There are no prescriptive rules as to what constitutes acceptable agreement. The engineer must exercise judgment based on the situation.*
>
> *Discrepancies between calculated and observed results can arise due to several causes. A common cause is an underestimation or overestimation in the intensity of ground shaking. To address this, the engineer might reconsider the reasonable range of ground shaking intensities and whether adjustments within this range can improve the agreement between observed and calculated damage results. Misestimation of shaking intensity is particularly critical when doing a linear analysis with a recorded ground motion with peaks and valleys in the response spectrum. See Section 3.4.2 for discussion of smoothing techniques that may help avoid sensitivities to these peaks and valleys.*
>
> *Another common cause is underestimating or overestimating structural stiffness values. In these cases, the engineer might consider adjustments to stiffness within a reasonable range. This includes considering the effects of soil-structure interaction, where some structural elements may be more sensitive to added flexibility due to foundation movements than others, which can affect the distribution of damage among the various elements.*
>
> *It may turn out that not only the magnitude of damage but also the distribution or type (mode or mechanism) of damage is not well represented by the structural analysis. In this case, it may be necessary to change the modeling approach, for example, changing from a linear elastic model to a nonlinear model. The mechanism analysis and study of irregularities, per Section 3.5, can also be used to inform the analysis refinement or confirm the need to use a nonlinear model.*

Particular attention should also be made to reconciling the inspection and analysis results at conditions that always require inspection, where the estimated Damage Class by analysis is significantly more severe than visually observed (i.e., DC2 per analysis, versus DC0 or DC1 per visual inspection). If the analysis cannot be reconciled with the visual observation, intrusive inspection (Section 3.8) of these locations should be considered if there are concerns about damage that cannot be detected by detailed visual inspection.

3.7.2 Reinspection

If revisions to the structural analysis result in identification of damage locations not already included in the detailed inspection, those new damage locations shall be inspected in accordance with Section 3.6.

Commentary: This completes the feedback loop from the updated analysis to identification of possible damage locations for detailed inspection per Figure 3-1.

3.8 Intrusive Inspection

Intrusive inspection (Section 2.2.2.3) shall be considered where little or no damage is observed by detailed visual inspection (Section 3.6.1) in locations where other indicators (Section 3.5) strongly suggest that damage is likely and cannot be reconciled. Intrusive inspection may also be warranted in components where visual damage is observed to determine the extent of damage. The details of the intrusive inspection shall be determined considering the structural material, the structural element, the nature of the observed damage, and the expected loads. Non-destructive testing (NDT) methods may be appropriate for performing such inspections, in accordance with Chapter 5. Locations requiring intrusive inspection can be reduced based on the guidance of Section 3.6.2.

Commentary: Intrusive inspections are generally limited to situations where there are strong reasons to suspect that there is damage that may not be detected through detailed visual inspection. In such cases, intrusive inspections can be used to identify whether structural damage is hidden behind the visible surface. Reasons to suspect that there is damage vary depending on the material and structural system but generally include: (1) instances where the earthquake demands calculated by analysis are well into DC2; (2) discrepancies between the damage classes calculated by analysis cannot be reconciled by judgements based on the expected system performance; and (3) experience from past earthquakes or laboratory tests indicate that hidden damage (not apparent from detailed visual inspection) is possible.

The details of an intrusive inspection will depend on the structural material and structural member. NDT may be an appropriate means of performing such inspections, including ultrasonic and penetrating radar techniques. Removal of intact structural materials should be a last resort and only done in exceptional circumstances. See Section 5.3.3 for material-specific intrusive inspection guidance.

Chapter 4: Evaluation of Earthquake Damage

4.1 Scope

This chapter provides in-depth guidance and essential commentary on the evaluation of earthquake damage to determine repair and retrofit needs. An earthquake-damaged building may be found to need repair to restore strength and deformation capacity (i.e., performance-critical repair). It may also be found to need both repair and retrofit. If performance-critical repair is not required, repair may still be warranted for other purposes (e.g., cosmetic, durability). If retrofit is not required, an engineer may still recommend retrofit to address pre-existing earthquake vulnerabilities and improve performance in future earthquakes.

4.2 General

As outlined in Figure 4-1, the evaluation of earthquake damage includes the following steps: (1) assignment of component damage classifications in accordance with Section 4.3 based on the damage identified in Chapter 3; and (2) determination of the Building Repair Outcome in accordance with Section 4.4. Principles for repair and retrofit design are provided in Section 4.4 and Section 4.5.

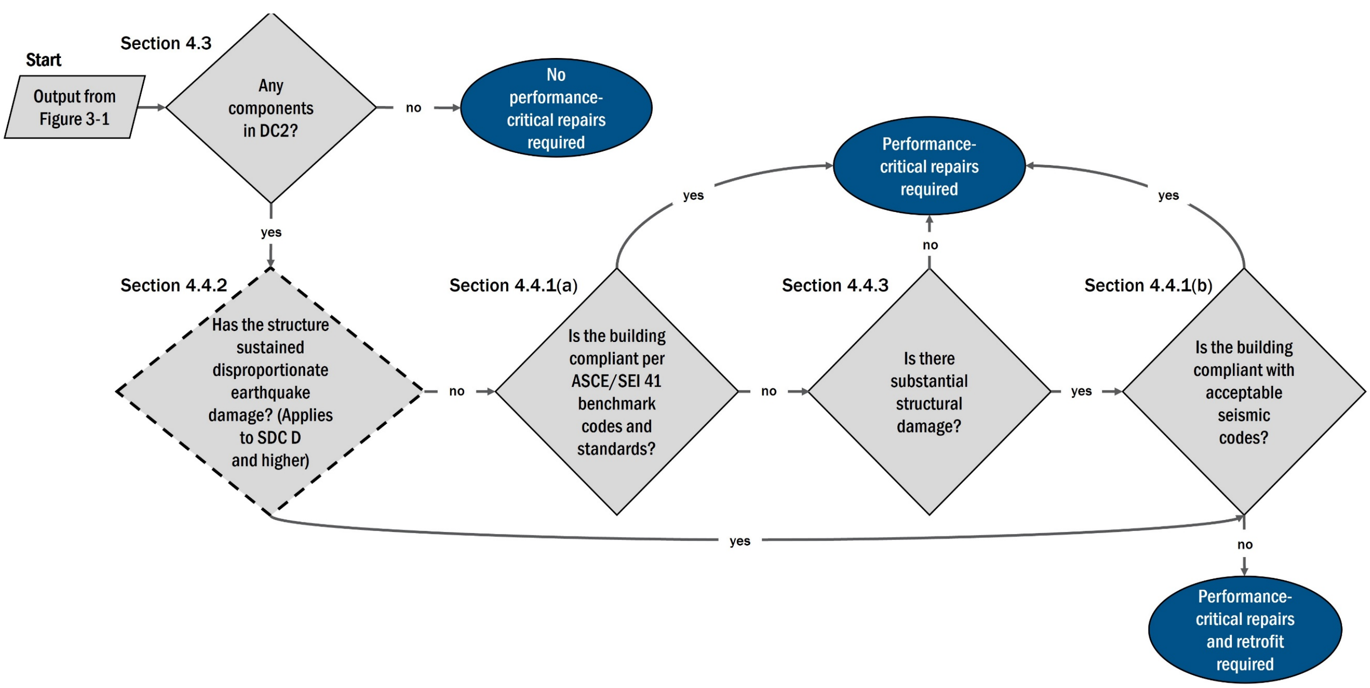


Figure 4-1 **Process for determining the Building Repair Outcome (adapted from Figure 2-4), with references to relevant sections of Chapter 4. Dashed lines indicate steps that might not be taken in all assessments.**

4.3 Component Damage Classification

Assign a component Damage Class in accordance with Table 4-1 and Figure 4-2. Damage Classes (Section 2.2.4) are based on comparison of observed damage with key visual damage features found in similar components that have Performance-Critical Damage, using the Visual Damage States (Section 2.2.5.1). If the review of Visual Damage States is not definitive, comparison of structural analysis results of the damaged building and the quantitative Performance-Critical Limits (Section 2.2.5.2) may also be used to inform this classification. Chapter 5 defines the Performance-Critical Limits (Section 5.4) and describes the Visual Damage States (Section 5.5) for reinforced concrete components.

Components inspected and found to be undamaged, and components not in inspection locations, shall be assigned DC0.

Components inspected and found to have damage consistent with deformations at or beyond the deformation at point DS2 in Figure 4-2 are defined as having Performance-Critical Damage (Section 2.2.2) and shall be assigned DC2. Chapter 5 also describes the identification of Performance-Critical Damage resulting from fatigue (Section 5.6).

All other components shall be assigned DC1.

A subset of components assigned to DC2 are identified as *gravity essential* in Section 5.8. These components have damage that could have high consequences in terms of loss of vertical-load-carrying capacity.

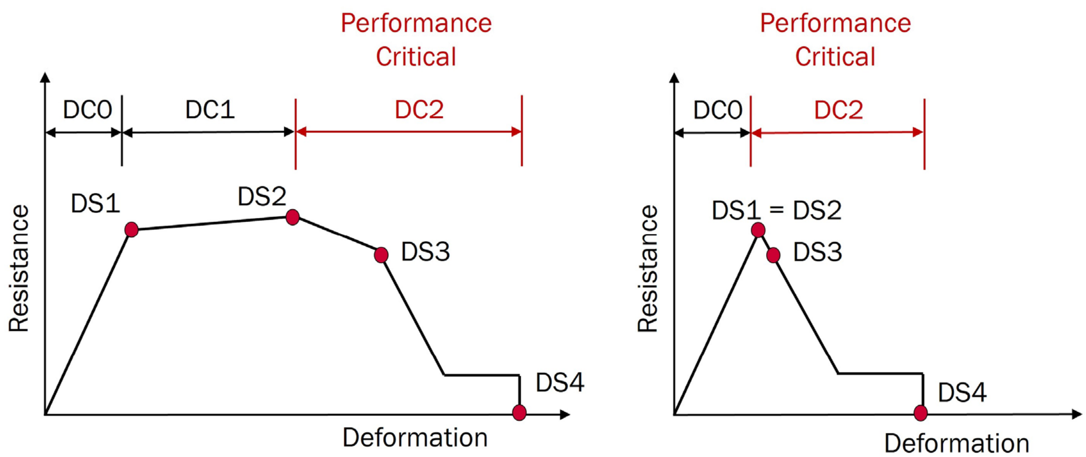
Table 4-1 Component Damage Classes

Damage Class	Relation to Component Damage States	Damage Description
DC0	Prior to DS1; Essentially elastic response	No Performance-Critical Damage; unlikely to require cosmetic, durability, or serviceability repairs
DC1	Between DS1 and DS2; Inelastic response without component strength loss	No Performance-Critical Damage unless indicated by other material-specific checks (e.g., fatigue damage), but cosmetic, durability, or serviceability repair may be warranted
DC2	Past DS2; Inelastic response with component strength loss	Performance-Critical Damage

> ***Commentary:*** *As described in Section 2.2.4, DC2 is the primary Damage Class of interest in these* Guidelines*. Components in DC2 impair the building's future earthquake performance. Accordingly, components in DC2 require repair to restore the strength and deformation*

capacity of the components to their pre-damage condition, i.e., they require performance-critical repair.

DC2 indicates impaired performance with respect to a future earthquake's lateral demands on the building. Components that are in DC2 may also lose vertical-load-carrying capacity; understanding whether the building is at risk of losing vertical-load-carrying capacity is significant for post-earthquake decision making. Components that carry substantial gravity and other vertical loads and that may lose vertical-load-carrying capacity shortly after losing lateral-load-carrying capacity are classified as gravity-essential components. Gravity-essential components are identified in Chapter 5. There are large uncertainties in the deformations at which components lose gravity-load-carrying capacity after component strength loss, so quantitative deformation limits are not appropriate for this purpose.



(a) Deformation controlled **(b) Force controlled**

Figure 4-2 Component Damage States (DS1-DS4) and Damage Classes (DC0-DC2) shown in relation to component backbone response.

4.4 Building Repair Outcome

Determine the Building Repair Outcome (Section 2.2.6) through the process illustrated in Figure 4-1.

If all components are in DC0 or DC1, no performance-critical repair is required to restore building strength and deformation capacity. For buildings with components in DC2, performance-critical repairs are required.

For some of the buildings needing performance-critical repairs, seismic retrofit may also be required. The determination of whether seismic retrofit is needed requires determination of: (1) whether the building complies with acceptable seismic codes (Section 4.4.1); (2) whether the building sustained disproportionate earthquake damage (Section 4.4.2); and (3) whether the building sustained substantial structural damage (Section 4.4.3).

Commentary: This section is used to determine whether a building needs repair or repair and retrofit. The flowchart in Figure 4-1 provides a suggested workflow to facilitate post-earthquake decision making, where the relevant sections of Chapter 4 are noted. The flowchart is organized with the quicker checks shown first, followed by more involved checks.

To identify buildings in need of seismic retrofit, these Guidelines *follow the intent of the IEBC, which aims "to identify especially vulnerable buildings at critical points in their useful lives and to require evaluation and possibly upgrade" (2018 IEBC, commentary to Section 405.2.2). In the IEBC, seismic retrofit is required for buildings that are found to be especially vulnerable based on observed earthquake damage. In the IEBC, this determination is made through evaluation of substantial structural damage and disproportionate earthquake damage. These* Guidelines *can be used to determine if substantial structural damage or disproportionate earthquake damage has occurred. More details are provided in Appendix D about how these* Guidelines *align with the intent of the IEBC. The engineer should also consult requirements of the Authority Having Jurisdiction if they differ from the IEBC.*

The building may be repaired or retrofitted even if it is not required by these Guidelines*. Depending on the building's pre-existing vulnerabilities, it may not necessarily have adequate performance in future earthquake shaking, even if it did not sustain observable earthquake damage or if the observed damage is not sufficient to lead to repair or retrofit according to these* Guidelines*.*

4.4.1 Compliance with Acceptable Seismic Codes

Determine whether the building complies with acceptable seismic codes. If the building is compliant with acceptable seismic codes, regardless of the level of damage, it is permitted to be repaired to its pre-damaged condition without retrofit.

4.4.1(A) COMPLIANCE PER ASCE/SEI 41 BENCHMARK CODES AND STANDARDS

As indicated by the workflow in Figure 4-1, these *Guidelines* first check compliance with reference to the benchmark building codes and standards defined in the most recent version of ASCE/SEI 41 Table 3-2, which addresses structural criteria for the Basic Performance Objective for Existing Buildings (BPOE). If the building satisfies the benchmark year/code for common building types in Table 3-2, including those for the height limits and seismic response parameter, the building may be deemed to comply with acceptable seismic codes and standards. In making this determination, the engineer should also review plans and documents to verify that design and detailing are consistent with the benchmark codes and standards. (Note: The seismic response parameter was added in ASCE/SEI 41-23 and is used for a comparison of the original design seismic response parameter and the current design value).

4.4.1(B) OTHER METHODS OF COMPLIANCE

Buildings not satisfying the benchmark codes and standards may also be shown to be compliant by demonstrating that if repaired to its pre-damaged state, the building would satisfy strength and drift criteria of the *International Building Code* (ICC, 2024c) using reduced seismic forces, taken as 75% of the prescribed seismic forces for new buildings by the Authority Having Jurisdiction. Values of R, Ω_o, and C_d used for analysis shall be in accordance with Chapter 12 of ASCE/SEI 7. To apply the values in Table 12.2-1 of ASCE/SEI 7, it should be demonstrated that the structural system will provide performance equivalent to that of an “Ordinary,” “Intermediate,” or “Special” system. Other resources (e.g., Hohener et al., 2018) may be used, subject to approval by the Authority Having Jurisdiction, to determine a value of R for systems that do not meet the criteria for a system in ASCE/SEI 7. A building satisfying the strength and drift criteria under these reduced seismic forces is deemed to be compliant with acceptable building codes and standards.

Compliance with acceptable seismic codes can also be demonstrated by evaluating whether the building, if repaired to its pre-damaged state, meets the ASCE/SEI 41 BPOE through Tier 3 evaluation procedures.

> Commentary: *Determination of compliance with reference to established benchmark codes and standards is intended to facilitate efficient post-earthquake decision making by first identifying buildings that are clearly compliant with acceptable codes and standards according to ASCE/SEI 41 BPOE criteria. This section is consistent with the IEBC in that it specifies determination of compliance by showing that the building meets ASCE/SEI 41 BPOE criteria. Use of benchmark codes and standards is a special case of the latter method. It is possible that buildings that do not satisfy compliance with benchmark years/codes may be shown to be compliant under the Tier 3 ASCE/SEI 41 procedures, so this level of analysis may be desirable for more complex cases, e.g. where substantial structural damage or disproportionate earthquake damage has occurred. Definition of reduced seismic forces as 75% of the prescribed seismic forces for new buildings by the Authority Having Jurisdiction is consistent with Chapter 3 of the IEBC and historical precedent in the assessment of existing buildings. Where retrofit is likely needed, an engineer may choose to focus on Tier 3 Evaluation Procedures, as these will also guide the retrofit design.*
>
> *These Guidelines do not address damage and repair of nonstructural components and systems and, as such, is concerned only with compliance of the building structure.*

4.4.2 Disproportionate Earthquake Damage

Evaluate a building in Seismic Design Categories D, E, or F to determine whether it has sustained disproportionate earthquake damage. Disproportionate earthquake damage is earthquake-related damage where *both* of the following conditions occur:

Condition 1

The 0.3-second spectral acceleration at the building site for the earthquake in question is less than 30% of the mapped acceleration parameter S_S.

Condition 2

> The vertical elements of the lateral-force-resisting system have suffered damage such that the lateral-force-resisting capacity of any story in any horizontal direction has been reduced by more than 10% from its pre-damaged condition.

The determination of reduction of lateral-force-resisting capacity of damaged buildings should follow the guidance described in Section 4.4.4 based on the damage classifications made in these *Guidelines*.

Disproportionate earthquake damage checks do not apply to buildings in Seismic Design Category A, B, or C.

As indicated by Figure 4-1, if the building is found to have sustained disproportionate earthquake damage, it is not permissible to evaluate compliance based solely on the benchmark code and standards due to the concern posed by the damage sustained in a small earthquake. In those cases, see Section 4.4.1(b).

> ***Commentary:*** *According to the IEBC, disproportionate earthquake damage "exists where a building has significant damage in even a very small earthquake. This damage is an indicator of severe damage, possibly collapse, in a larger event" (IEBC, 2018, Commentary to Section 405.2.2). The text of Conditions 1 and 2 above is taken directly from the IEBC.*
>
> *Condition 1 provides a check on the level of ground shaking at the building site using a comparison between the observed spectral accelerations and mapped acceleration parameter S_S. The 30% value is adopted directly from the IEBC.*
>
> *For Condition 2, the system considered should include any elements that constitute the lateral-force-resisting system. If the lateral-force-resisting system is not clearly defined, all components contributing to lateral resistance shall be included. It is recommended that the condition of diaphragms be considered.*
>
> *In assessing disproportionate earthquake damage, the evaluating engineer should form a view as to whether the damage observed is consistent with the expected modes of inelastic deformation of the structural system in question. For example, for modern moment resisting frames, inelastic deformations are expected primarily in flexural yielding of beam ends, yielding from flexure/axial at the base of the columns, and limited yielding elsewhere in columns. Significant shear damage or failures in beams or columns or joints would be inconsistent with the desired modes of inelastic deformation. This type of damage may indicate that the building is not performing as expected and may warrant further engineering investigation and possibly the need for retrofit in addition to repair.*
>
> *Per the workflow in Figure 4-2, if disproportionate earthquake damage occurs, then an assessment of the building per ASCE/SEI 41 beyond benchmark codes and standards is required to determine compliance. Disproportionate earthquake damage warrants careful consideration of the structure and its design and detailing.*
>
> *For more detail on disproportionate earthquake damage, see Appendix D.*

4.4.3 Substantial Structural Damage

A building shall be evaluated to determine whether it has sustained substantial structural damage, which is defined as earthquake-related damage where *either* of the following conditions occur:

Condition 1

The vertical elements of the lateral-force-resisting system have suffered damage such that the lateral-force-resisting capacity of any story in any horizontal direction has been reduced by 33% or more from its pre-damaged condition.

The determination of reduction of lateral-force-resisting capacity of damaged buildings should follow the guidance described in Section 4.4.4, based on the damage classifications made in these *Guidelines*.

Condition 2

Substantial damage observed to gravity-essential components indicating that the capacity of vertical components carrying gravity load has been substantially reduced from its pre-damaged condition. If gravity-essential components in DC2 support more than 30% of the load at the roof or any individual floor, the building is deemed to have sustained substantial structural damage. Gravity-essential reinforced concrete components are identified in Section 5.8.

Commentary: The IEBC uses the concept of substantial structural damage to identify those buildings that by policy should be retrofitted. In making a determination of whether substantial structural damage has occurred, the cause of the damage is irrelevant. That is, damage from any source, even if unrelated to the earthquake damage, is considered substantial structural damage if it reduces the capacity of the lateral-force-resisting or gravity-load-carrying systems at or above the amounts noted.

These Guidelines' *definition of substantial structural damage is consistent with the definition of substantial structural damage in the IEBC in that it consists of a check on loss of lateral-force-resisting capacity (Condition 1) and a check on damage to vertical-load-carrying capacity (Condition 2). This section also clarifies how the damage classifications in this document can be used to determine whether substantial structural damage exists.*

Section 4.4.4 provides guidance on how to calculate the loss of capacity of the lateral-force-resisting system in alignment with the metrics associated with the inspections and analyses in these Guidelines*. This guidance is intended to facilitate post-earthquake decision making, as the IEBC provides no specific rules on these calculations.*

While nonstructural damage can indicate the possible presence of structural damage, nonstructural damage is not considered in the determination of substantial structural damage.

For more detail on substantial structural damage, see Appendix D.

4.4.4 Reduction in Lateral-Force-Resisting Capacity

The determination of disproportionate earthquake damage and of substantial structural damage may necessitate structural analysis to quantify the reduction in lateral-force-resisting capacity of a damaged building relative to its pre-damaged condition.

To calculate the capacity of the damaged building, any components that are assigned to DC2 shall be assumed to have a reduced strength. This reduced strength shall be taken as zero, unless it can be shown through nonlinear analysis that the deformation demands on the component in the damaging earthquake did not exceed the Collapse Prevention (CP) acceptance criterion for that component, as defined in ASCE/SEI 41. Where the CP acceptance criterion has not been exceeded in a component, its residual strength can be assessed using the residual strength ratio, *c*, defined in ASCE/SEI 41 modeling parameters. This calculation should consider all elements that are part of the lateral-force-resisting system, or all the structural elements in the building if there is no clearly defined lateral system.

Once the reduced strengths of the damaged components have been established, the reduction in lateral-force-resisting capacity of the damaged building structure can be determined through one of the following approaches:

- By comparing a summation of component strengths, accounting for differences in rigidity and deformation compatibility, for the pre-damaged and damaged building structure in each story where damage has occurred;
- Using an ASCE/SEI 41 Nonlinear Static Procedure, or a mechanism analysis as described in FEMA P-2018, where maximum lateral (base shear) strength is compared for models of the pre-damaged and damaged building structure; or
- Using an ASCE/SEI 41 Linear Static Procedure, where the lateral-force-resisting capacity of the pre-damaged and damaged building structure are determined by comparing the equivalent lateral forces (base shears) at which the most critical component of the structure reaches a demand corresponding to the CP acceptance criteria for primary members.

> ***Commentary:*** *The estimates of lateral-force-resisting capacity for both the pre-damaged and damaged states should represent expected values, including expected material properties. Damage to the diaphragm and associated elements should be evaluated to determine if this damage compromises the lateral-force-resisting capacity.*

4.5 Repair

Building repairs may be undertaken to restore component and building strength and deformation capacity, i.e., performance-critical repairs. These repairs may also be made to address cosmetic, durability, or serviceability problems produced by earthquake damage. Chapter 5 describes repair techniques for reinforced concrete.

4.5.1 Repair to Address Performance-Critical Damage

Performance-critical repair to restore structural components to their pre-earthquake condition is required for components in DC2. These performance-critical repairs may occur in conjunction with repairs to restore appearance, durability, or serviceability.

> Commentary: *This type of repair is intended to restore strength and deformation capacity of the damaged components. This type of repair may also restore some stiffness. Details of the repair options that may restore or improve strength and deformation capacity are provided in the materials chapters, e.g., Section 5.9.*
>
> *Performance-critical repairs may include local modifications of components if it can be demonstrated that the modification is able to restore or improve the performance of the component, and does not detrimentally affect the rest of the structure (i.e., does not change the expected inelastic mechanism). An example of such a repair would be wrapping or jacketing a concrete column with shear damage.*

4.5.2 Repair to Restore Appearance or Durability

Repairs to restore the appearance, durability, or both of damaged components are generally permitted, either as the sole repair objective for components in DC0 or DC1, i.e., where performance-critical repair is not required, or in conjunction with performance-critical repairs.

> Commentary: *This type of repair may include patching, sealing, and painting to improve visual appearance or to improve weather protection or fire protection. Structural benefits of these repairs typically are negligible, and these repairs are not required to restore strength, deformation capacity, or stiffness. More details are provided in Section 5.9.1.*

4.5.3 Repair to Achieve Alternative Performance Objectives

A building owner or other responsible stakeholder may choose repairs to achieve alternative performance objectives, provided those objectives are not less than the performance objectives mandated by local requirements. An example is structural repair to achieve a serviceability performance objective that is intended to limit drift demands in a future event. Such repairs will commonly be accompanied by additional structural analysis to demonstrate that the serviceability performance objectives are achieved.

> Commentary: *Repairs required to achieve alternative performance objectives will be highly structure dependent. One example of an alternative performance objective is the PEER serviceability criteria for tall buildings (PEER, 2017), which limit drifts to 0.5% under a service level earthquake. Serviceability-related objectives may be difficult to achieve in a damaged building, as many repair actions do not fully restore stiffness. In addition, some structures may not achieve these objectives in the pre-earthquake condition.*

4.6 Retrofit

Seismic retrofit may need to be undertaken if the structural damage is severe, as described in Section 4.4. Additionally, a building owner or other responsible stakeholder may choose seismic retrofit to address pre-existing vulnerabilities or to achieve an alternative seismic performance objective. Seismic retrofit of a damaged building will likely be accompanied by repair.

Seismic retrofit should be designed in accordance with the provisions of ASCE/SEI 41. Structural analysis for the design of the seismic retrofit in a post-earthquake context shall be in accordance with ASCE/SEI 41, except as modified in this section. Guidance for modeling repaired and damaged reinforced concrete components is provided in Chapter 5. Where the assessment of low-cycle fatigue indicates that the damaging earthquake may have reduced the deformation capacity or fatigue resistance in future earthquakes, these effects should be considered in the structural analyses and performance assessment criteria for design of the retrofit.

> Commentary: *Seismic retrofit can be used to improve performance, improve confidence in future performance, or both. Retrofit design, as described in ASCE/SEI 41 and other reference documents, requires reanalysis and consideration of the entire structural system, which may affect repair actions. For example, some DC2 components may not need to be repaired if they are not relied upon in the retrofitted structural system. Additional guidance techniques for seismic rehabilitation are provided in* FEMA 547, Techniques for the Seismic Rehabilitation of Existing Buildings (FEMA, 2006)*.*
>
> *ASCE/SEI 41 contains provisions for the analysis of existing buildings, including general analysis requirements, procedures for selecting among various analysis methods, specific analysis requirements, component acceptance criteria, and procedures for developing alternative modeling parameters and acceptance criteria.*
>
> *Performance-Critical Damage that has occurred in the damaging earthquake may result in critical strength loss that will affect performance in subsequent earthquakes and retrofit design. Such damage, however, is generally repaired to restore its strength, such that the strength loss need not be considered when evaluating the retrofitted building. However, in some cases the retrofit performance objective may involve deformation demands that are less than those that occurred in the damaging earthquake. In this case, the prior earthquake damage may cause a reduction in the effective stiffness of the structural system, which may lead to an increase in building vibration period and an increase in peak displacement response. These effects should be considered by modifying the effective stiffness of the components. It may also be appropriate to adjust the effective damping. See Section 5.7.*

Chapter 5: Reinforced Concrete

5.1 Scope and Organization

This chapter provides guidance for assessment of cast-in-place reinforced concrete beam-column frames, slab-column frames, and walls with or without coupling beams subjected to earthquake ground shaking. The chapter is organized in a series of sections addressing aspects of earthquake damage assessment and repair specific to reinforced concrete, as follows:

- Section 5.2 provides an overview of reinforced concrete structural systems addressed in these *Guidelines*, including guidance on typical damage and behavior modes.
- Section 5.3 provides guidance on visual inspection, including assessment of pre-existing conditions, conditions that warrant intrusive inspection, and conditions that always require inspection.
- Section 5.4 provides performance-critical force and deformation limits for reinforced concrete components, which are used for determination of possible damage locations (Section 3.5.5) and reconciliation of inspection and structural analysis (Section 3.7).
- Sections 5.5 and 5.6 define whether a component has experienced Performance-Critical Damage and requires repair. Section 5.5 provides guidance on determining the Damage Class of a damaged component, including the use of Visual Damage States (Section 2.2.4.1). Section 5.6 provides guidance on assessment when repair is necessary to address low-cycle fatigue damage.
- Section 5.7 presents modeling parameters for damaged components.
- Section 5.8 provides reinforced concrete-specific criteria used to define substantial structural damage.
- Section 5.9 describes repair techniques and guidance for modeling components after repair.

5.2 Structural Systems Behavior

5.2.1 Considered Systems and Typical Damage Locations

Table 5-1 summarizes damage locations and types for various reinforced concrete structural elements considered in these *Guidelines*. See Section 5.2.2 through Section 5.2.6 for more detailed descriptions. Damage locations may also be influenced by vertical and horizontal irregularities in the structural framing. Evaluation for damage should consider these elements, components, damage modes and locations, and possible effects of irregularities, as applicable. Table 5-1 should not be considered exhaustive; other damage not listed in Table 5-1, if applicable, should also be evaluated.

The criteria in this chapter can be used for reinforced concrete frame and wall components even if some elements in the building, such as a wood diaphragm, are not reinforced concrete.

Table 5-1 Structural Element Damage Locations

Structural Element	Components	Primary Observed Damage Mode	Typical Damage Location
Structural wall	Wall	Flexure	Adjacent to critical sections
		Shear	Critical sections and stories
		Shear friction	Construction joints
	Coupling beam	Flexure	Member ends
		Shear	Along member length
	Panel zone	Shear, spalling	Within panel zone
Beam-column frame	Beam	Flexure	Within beam, especially near connection to column
		Shear	Within beam, especially near connection to column
	Column	Flexure	Within column adjacent to column-joint interface
		Shear	Within column at any point along length
	Joint	Shear	Within joint
Slab-column frame	Slab	Punching shear	Slab-column connection region within slab
		Slab yielding	Within slab adjacent to columns, possibly extending across slab panel
Diaphragm	Collectors	Tension, compression, shear friction	Diaphragm to moment frame and wall connections
	Chords	Tension or compression	Diaphragm edges
	Distributed	Shear	Diaphragm span
Foundation	Shallow foundations	Punching shear	Connections between columns or wall boundaries and shallow foundation element
		Flexure	Foundation element critical sections
		Shear	Foundation element critical sections
	Deep foundations	Pile cap damage	Within pile cap
		Pile damage	Pile-to-cap connection

5.2.2 Structural Walls

Structural walls in reinforced concrete buildings considered in these *Guidelines* are made of cast-in-place reinforced concrete and include cantilever walls, pier-spandrel walls including coupled walls, and other walls such as perforated walls.

> Commentary: *The elevations of Figure 5-1 illustrate three general categories of concrete wall element configurations. Cantilevered wall elements are those that act predominantly as vertical beam-columns restrained at their foundation level. Pier-spandrel elements, including coupled walls, are those with a generally regular pattern of openings that form a configuration of vertical wall segments (piers) and horizontal wall segments (spandrels or coupling beams). Other wall elements can include mixed configurations or walls with large, irregular perforations. Wall elements are commonly coupled with beam-column or slab-column moment frames, in which case the walls and frames are considered as individual elements that interact because of their interconnection ensuring deformation compatibility.*
>
> *Identification of component types in concrete structural wall elements depends, to some degree, on the geometry and relative strengths of the wall segments. Vertical segments (bounded laterally by openings or edges) are sometimes referred to as wall piers, whereas horizontal segments (bounded vertically by openings or edges) are sometimes referred to as coupling beams or spandrels. Table 5-2 presents the anticipated damage by component type for reinforced concrete walls.*

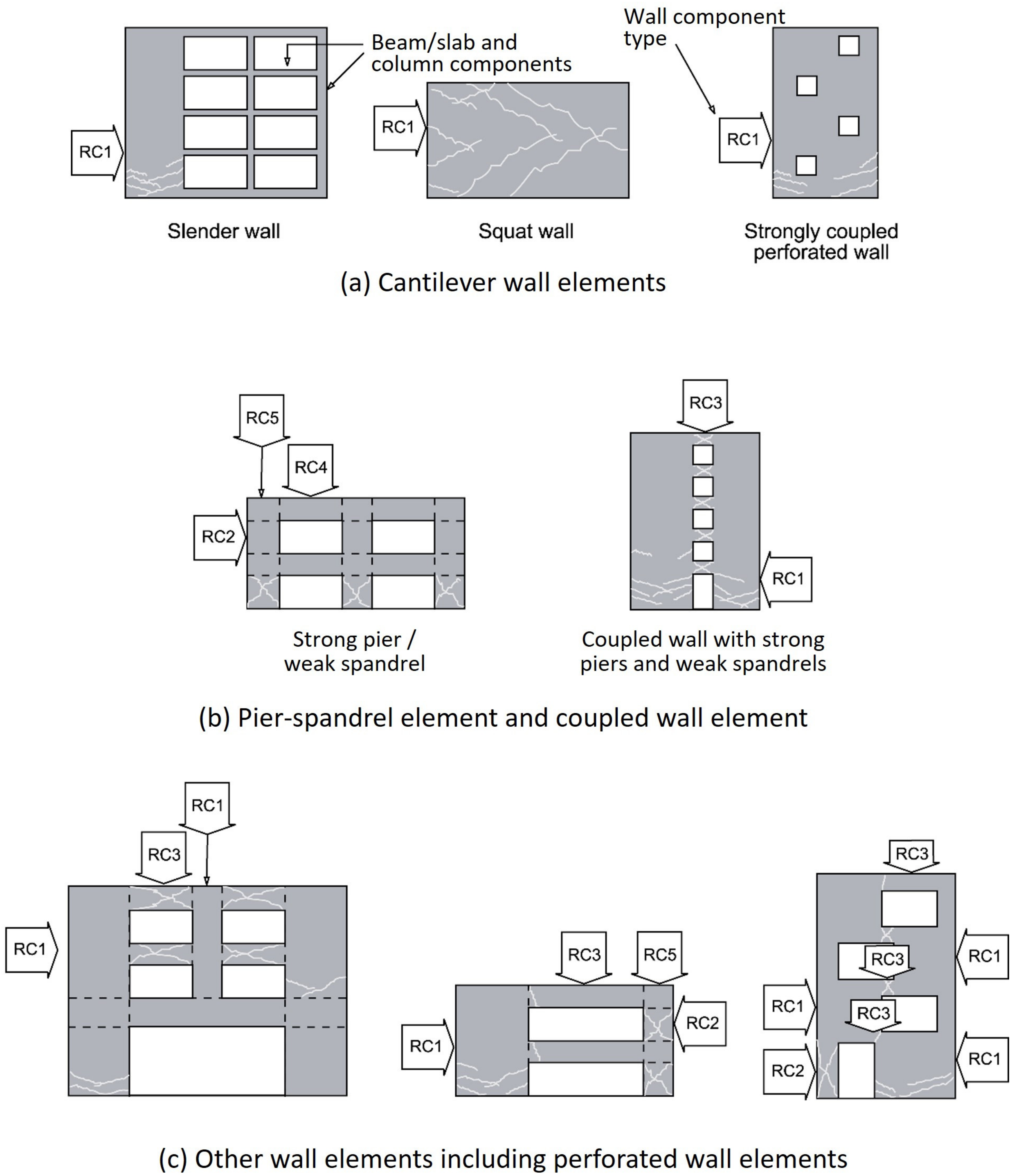


Figure 5-1 Illustration of different types of structural wall configurations, wall components, and anticipated damage patterns. See Table 5-2 for descriptions of the various wall component types (e.g., RC1) illustrated in the figure (credit: adapted from FEMA 306).

Table 5-2 Component Types and Anticipated Damage for Reinforced Concrete Walls [1]

Component type		Description	ASCE/SEI 41 Designation
RC1	Cantilever wall or stronger wall pier	This type of component is stronger than beam or spandrel components that may frame into it, such that nonlinear behavior (and damage) is generally concentrated at the base, with a flexural plastic hinge or shear damage. Includes isolated (cantilever) walls. If the component has a major setback or reduction of reinforcement above the base, this section should be also checked for nonlinear behavior.	Monolithic reinforced concrete wall or vertical wall segment
RC2	Weaker wall pier	This type of component is weaker than the spandrels to which it connects. Damage is characterized by flexural hinging top and bottom or shear damage.	
RC3	Weaker spandrel or coupling beam	This type of component is weaker than the wall piers to which it connects. Damage is characterized by hinging at each end, shear damage, or sliding shear damage.	Horizontal wall segment or coupling beam
RC4	Stronger spandrel	This type of component should not sustain damage because it is stronger than attached wall piers. If this component is damaged, it should be reclassified as RC3.	
RC5	Pier-spandrel panel zone	This component is a pier-spandrel connection zone, similar to the connection between a beam and column in a beam-column frame. Typically, not a critical damage area in reinforced concrete walls.	Wall segment

[1] Reinforcement anchorage or splice failures can occur anywhere that insufficient anchorage or lap length is provided within a wall, but this damage mode is not specifically called out in Table 5-2.

Where walls intersect to form L-shaped, T-shaped, C-shaped, or similar cross sections, the entire cross section typically is considered as an integral unit and a single element/component. The contribution of flanges and wall returns should be considered in evaluating the behavior of the element/component using guidance given in ASCE/SEI 41. Note that for loading in one principal direction of a flanged wall, part of the wall acts as the web and part acts as the flange, whereas for loading in the orthogonal direction the portions considered as webs and flanges switch. Observed damage to a flanged wall may be the result of horizontal loading in either or both directions.

5.2.3 Beam-Column Frames

Beam-column frames in reinforced concrete buildings considered in these *Guidelines* are made of cast-in-place reinforced concrete.

> *Commentary: Figure 5-2 illustrates a representative beam-column frame. Three main components occur in beam-column frames: Beams, columns, and beam-column joints. In most cases, the beams will also support and be monolithically cast with slabs that support gravity loads and provide diaphragm action. These* Guidelines *were mainly developed around frames in which the columns and beams have relatively compact sections (depth to width ratios not exceeding 2.5/1), although these* Guidelines *should be applicable to beam-column frames with cross sections outside this range. Additionally, these* Guidelines *were mainly developed considering beams, columns, and joints that have limited eccentricity, in which the centerline of the beam is located within the width of the column and at least some of the beam longitudinal reinforcement passes within the column core (as defined by the boundaries of the column longitudinal reinforcement). These* Guidelines *can be applied to frames not meeting this limitation on eccentricity, but the additional effects of eccentricity on behavior should be considered by the evaluating engineer.*

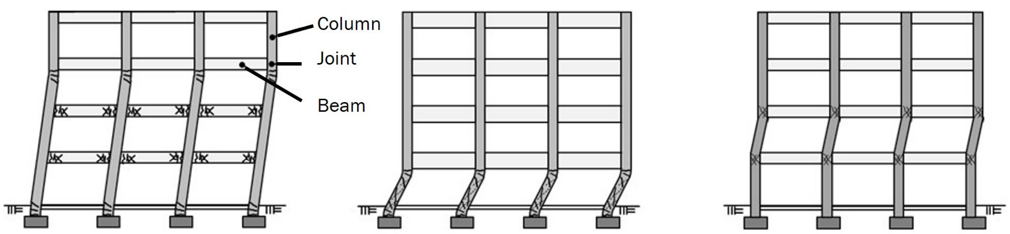


Figure 5-2 Illustration of representative beam-column frame configuration and anticipated inelastic mechanisms (credit: adapted from Moehle, 2015).

> *Figure 5-2 also illustrates some damage and inelastic mechanisms to anticipate when inspecting a beam-column frame following earthquake shaking. Beams and columns can experience damage associated with flexure or shear, while joints may experience diagonal cracking associated with joint shear. Splitting or other failures associated with high bond stress, or anchorage or splice failure, can occur in beams, columns, or joints.*
>
> *If columns are generally stronger than beams and are controlled by flexural response, one might anticipate an inelastic mechanism forming in the lower levels and extending over multiple stories above the lower levels (Figure 5-2a). The height of the apparent yielding mechanism will depend on the relative strength ratios of the columns and beams. If the columns are generally weaker than the beams or the column actions are limited by shear failure, then it is more likely that a story mechanism will form in which the predominant inelastic deformation concentrates in a single story, typically but not always near the base of the frame (Figure 5-2b). Finally, a weak-joint frame is one in which multiple joints in adjacent*

levels have strengths less than the joint shear demands from the beams and columns framing into the joint. Joint strength can degrade rapidly, such that the columns spanning between failed joints lose their ability to transfer moment through the floor system (Figure 5-2c) – the effect is essentially the same as the effect of column failure in a strong-beam/weak-column frame (Figure 5-2b).

Although the above description and Figure 5-2 indicate that damage will likely be concentrated on the lower stories where demands are typically highest, the location of damage will depend on relative stiffness and strength changes over the height of the beam-column frame, as well as higher-mode effects. Frames with setbacks (more bays on lower stories) or significant changes in column or beam strength and stiffness over the height of the building may be prone to damage initiating from the top of the stiffer portion of the beam-column frame.

5.2.4 Slab-Column Frames

Slab-column frames in reinforced concrete buildings considered in these *Guidelines* are made of cast-in-place reinforced concrete, with or without capitals, shear reinforcement, or post-tensioning.

Commentary: *Figure 5-3a illustrates a representative slab-column frame. The main components of interest include the slab (and its connection with the columns), slab drop capitals, and columns.*

Figure 5-3b is a plan view illustrating representative damage and inelastic mechanisms to anticipate when inspecting a slab-column frame following earthquake shaking. Damage will usually be concentrated in the slabs, possibly including slab flexural yield lines and slab punching shear failure at the connection between the slab and the column. Yield lines may extend across the full slab width or may be concentrated locally adjacent to column or capital faces. Slab punching can extend partly or entirely around an individual connection and may involve vertical movement as the slab separates from the column or capital and drops perceptibly. In slabs with continuous bottom reinforcement or draped post-tensioned tendons passing over the column, vertical movement of the slab may be arrested by the reinforcement after punching failure. Without that reinforcement, a punching failure at one connection may progress to adjacent connections with the possibility of partial or complete floor failure. Columns are often stronger than the slabs framing into them, so Performance-Critical Damage to columns is normally not anticipated, but it can occur where the columns are relatively weak compared with the slabs.

A slab-column joint is defined as the part of the column (including a column capital if present) within the depth of the slab (including drop panel or shear cap), similar to the definition for a beam-column joint. Slab-column joints (not to be confused with slab-column connections described above) generally do not sustain damage due to earthquake shaking and, therefore, are not considered further here.

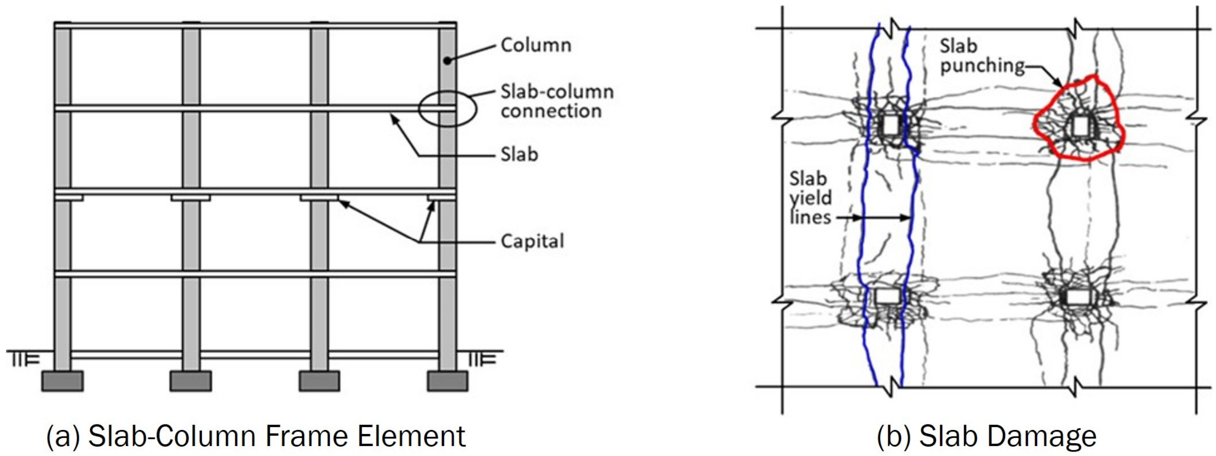


Figure 5-3 Representative slab-column frame configuration and slab damage (credit: adapted from Moehle, 2015).

5.2.5 Diaphragms

Diaphragms in reinforced concrete buildings considered in these Guidelines are made of cast-in-place reinforced concrete, cast-in-place composite topping slab on a precast floor or roof, cast-in-place noncomposite topping on a precast floor or roof in which the topping slab is designed to provide full diaphragm action, or interconnected precast floor or roof members without cast-in-place topping. See requirements in ACI 318, *Building Code Requirements for Structural Concrete and Commentary* (ACI, 2019).

> *Commentary: Diaphragm components of interest include: (1) chords acting in tension or compression to resist in-plane diaphragm moment; (2) diaphragm segments resisting in-plane shear; and (3) collectors that transfer distributed shear between the diaphragm and the vertical elements of the lateral-force-resisting system.*
>
> *Diaphragm chords can sustain damage associated with reinforcement tension splice failure. Also, inadequately supported chord reinforcement can buckle out of plane when loaded in compression, and this tendency may be exacerbated if the bars have residual tensile strain from previous tension loading cycles. This problem can be exacerbated where chord bars are located within relatively thin topping slabs.*
>
> *The field of the diaphragm may be susceptible to in-plane shear damage that may include diagonal cracking. Topping slabs sometimes are reinforced with welded-wire mesh, which may have relatively small fracture strain that could result in localized failures. Shear damage can also occur in the form of slip at interfaces between a diaphragm and vertical elements of the lateral-force-resisting system due to either high interface shear or inadequately prepared interfaces.*
>
> *Collectors may sustain tensile yielding or failure at splices and at connections with vertical elements of the lateral-force-resisting system. Fracture at connections with wall boundaries*

may be especially vulnerable because wall uplift can create a "kink" in the collector bars at this location.

Diaphragm damage may also be prone to occur at discontinuities, such as vertical steps, diaphragm openings, and diaphragm reentrant corners.

Diaphragms with precast floor units, with or without topping slabs, can sustain damage due to beam elongation resulting in wide cracks between units and beams at the corners of the diaphragm. The seating for the precast units can also sustain damage due the relative rotation between the support beam and the precast unit. For hollow-core floor units, web cracking can occur at relatively low drifts. This damage, internal to the hollow-core unit, can be difficult to detect after an earthquake and can affect the gravity-load-carrying capacity of the unit. While recognizing the importance of inspecting precast diaphragm units for damage and assessing associated potential loss of gravity-load-carrying capacity, these Guidelines *do not provide specific criteria for judging the residual capacity.*

5.2.6 Foundations

Evaluation of a building for earthquake damage effects includes consideration of damage to the foundation. A wide variety of foundations for reinforced concrete buildings may be encountered; therefore, only general guidance is provided in these *Guidelines*. Foundations in reinforced concrete buildings considered in these *Guidelines* are made of cast-in-place reinforced concrete with or without piles.

The necessity for repair of damaged foundation elements should be judged based on consideration of the effect of the damage on performance of the building in future earthquakes. In some cases, damage may be accepted without repair if its effect on future performance is deemed to not be significant.

Commentary: Structural information to be gathered for a foundation includes: foundation type; foundation configuration, including dimensions and locations; and material composition and details of construction. Damage to foundation elements can occur due to movement of the supporting soil, overstress of structural foundation elements, or damage to connections between the vertical elements of the lateral-force-resisting system and the foundation elements.

Damage from movement of the supporting soil may be due to fault rupture; liquefaction and associated movement possibly including lateral spreading, settlement, bearing capacity failure, and flotation; differential settlement; compaction; and landsliding. These effects will commonly be evident in the soil surrounding the building and foundation or differential lateral or vertical movement of foundation elements possibly accompanied by movement-induced damage to the foundation elements themselves or to the vertical or horizontal elements of the structural system that they support.

Shallow concrete foundations include spread footings, strip footings, combination footings, and mat foundations. They may be reinforced for bending moments arising from gravity loads or, especially in newer buildings, may be reinforced also considering bending moments arising from lateral loads. Shear reinforcement is not always used. Observation of cracks without significant vertical offsets may be indicative of flexural cracking, whereas observation of cracks with vertical offsets may be indicative of shear damage.

Deep concrete foundations include driven or cast-in-place concrete piles, often interconnected by pile caps or mats. Driven piles can be concrete, steel shapes, steel pipes, or composite concrete in a driven shell. Common damage locations can include connections between piles and pile caps due to the transfer of vertical compression or tension, moment, or shear. Damage can also occur to piles at depth due to changes in lateral stiffness of soil strata. Damage to piles and their connections to pile caps or mats are generally not visible by inspection. They might be suspected where settlement of a piled foundation is observed.

Connections between vertical elements of the lateral-force-resisting system and foundation elements can be subjected to high compressive, tensile, and moment-induced forces. A common primary concern is compression punching shear or tension breakout failures where columns or wall boundary elements connect to the foundation.

5.2.7 Systems Not Explicitly Considered

These *Guidelines* do not explicitly consider reinforced concrete structural systems not specifically identified in Sections 5.2.2 through 5.2.6. These include, but are not limited to:

- Structural walls made of precast elements with weak connections,
- Frames made of precast elements with weak connections,
- Frame and wall components of composite construction such as concrete encased steel sections,
- Reinforced concrete frames with masonry infill walls, and
- Slab-column frames made of precast slab panels and lift-slab construction.

Commentary: *Although the above systems and components were not explicitly considered in the development of these* Guidelines *and should be assessed with caution, some of the guidance and overall assessment process may be useful in guiding inspection and determining if repair is necessary. For example, columns and beams made of concrete-encased steel sections may still exhibit some of the damage states described herein, but focus should be on identifying buckling and fracture of the steel sections.*

These Guidelines *were not developed to provide comprehensive guidance for infilled frames. In particular, the inspection triggers and the details of damage to infills (either masonry or concrete infills) and to the framing members surrounding the infills have not been developed* *considering infilled frames. Nonetheless, these* Guidelines *can provide useful guidance for inspecting and assessing earthquake-induced damage to infilled frames. An engineer using these* Guidelines *to assess earthquake-induced damage in infilled frames will need to use judgment to consider the distinct behavioral differences associated with the infills and infill-frame interaction. The failure mode of columns in infilled frames can be influenced by the strength of the infill and the presence of gaps between the infill and the frame. Such interaction can also occur with stiff nonstructural components, such as precast cladding, without sufficient allowance for movement between the column and the nonstructural component. It is recommended that columns in infill frames, or with possible interaction with stiff nonstructural components, be assessed assuming the column may be either shear-controlled or flexure-controlled (referred to as "conforming" in Table 5-3).*

5.3 Inspection

5.3.1 Visual Inspection

Conduct a preliminary inspection in accordance with Section 3.3 or a detailed visual inspection in accordance with Section 3.6.1.

Particular attention should be paid to visually identifying any severe damage states commonly associated with initiation of component strength loss and possibly necessitating performance-critical repair. Such damage states are summarized in Section 5.5.1.

Commentary: For reinforced concrete construction, detailed visual inspection will include viewing the concrete surface to identify and quantify cracking and crack widths, delamination, spalling, crushing, development/anchorage failures, and bar buckling and fracture. See Section 5.5.1 for specific examples of relevant damage for identifying if performance-critical repair is necessary.

5.3.2 Pre-existing Conditions

Assess if any observed damage is from pre-existing conditions in accordance with Section 3.3.4.

Commentary: Pre-existing damage can be due to drying shrinkage, gravity loads including construction loads, a previous earthquake, or other effects. A main purpose in identifying pre-existing damage is because doing so may help to identify the source of the observed damage and thereby help the engineer to understand how the building has responded to various loads it has seen in the past, including the damaging earthquake that has triggered the inspection. Pre-existing damage should not be ignored when assessing the current damage state of the building but, instead, should be incorporated in that assessment.

Because the evaluation of earthquake-damaged buildings is typically conducted within weeks or months of the event, cracking and spalling of structures caused by earthquakes is normally relatively recent damage. Cracks associated with drying shrinkage, gravity loads, or *a previous earthquake, on the other hand, would be relatively old. ACI PRC-224.1-07,* Causes, Evaluation, and Repair of Cracks in Concrete Structures *(ACI, 2007), discusses possible causes of cracking in reinforced concrete. General guidance for assessing the relative age of cracks based on visual observations is given below.*

Recent cracks typically have the following characteristics:

- *Small, loose edge spalls,*
- *Light, uniform color of concrete or mortar within crack,*
- *Sharp, uneroded edges, and*
- *Little or no evidence of carbonation.*

Older cracks typically have the following characteristics (depending on exposure):

- *Paint, adhesive materials, or soot inside crack,*
- *Water, corrosion, or other stains seeping from crack,*
- *Previous, undisturbed patches over crack,*
- *Rounded, eroded edges if in a location where edges are subject to wear, and*
- *Deep carbonation.*

5.3.3 Intrusive Inspection

Conduct intrusive inspection as required by Section 3.6.2 where it is suspected that (1) damage to the cover concrete in the form of spalling, crushing, or inclined or bond-splitting cracking may extend into the core concrete, or (2) reinforcing bars may have fractured inside otherwise intact concrete. At least the following shall be considered:

- Where cover concrete is spalled, crushed, or delaminated, consider selective removal of damaged cover concrete to evaluate the extent to which the surface damage extends into the concrete core and whether bar buckling has occurred.

- Where cover concrete shows inclined cracks wider than 2 mm (about 1/16 in.), selective removal of cover concrete should be considered to evaluate the extent to which the inclined cracking extends into the core.

- Where the concrete cover at a lap splice shows bond-splitting cracking and transverse reinforcement conforming with ACI 318 is not provided, selective removal of cover concrete should be considered to evaluate the extent to which the cracking has impacted the likely performance of the splice.

- Where flexure-dominated cracking at critical sections for moment is suspected or observed in members with flexural reinforcement less than the minimum reinforcement currently required by ACI 318, consider selective removal of concrete cover to expose longitudinal reinforcement at the extreme flexural tension zone of moment critical sections to identify whether that reinforcement has fractured.

Commentary: If cover concrete is visibly damaged, it may be advisable to selectively remove the damaged cover concrete to expose whether surface damage extends into the core concrete or whether reinforcement is damaged. Sometimes a concrete component may be covered in plaster as part of nonstructural fit-out; such plaster should be removed to confirm if the cracking extends into the structural component.

Note that concrete cover generally should not be removed unless already spalled, loose, or showing inclined cracks (larger than 2 mm, or about 1/16 in.) associated with shear; if the concrete cover is undamaged, the core and bars can also be assumed to be undamaged. An exception, however, occurs in some components with low longitudinal reinforcement ratios (i.e., less than that required to resist the cracking moment). In such members, fracture of tension reinforcement may occur at a single flexural crack that may not be obvious through visual inspection because the single crack closes under gravity load stresses following the earthquake.

The minimum reinforcement to be considered from ACI 318 is the minimum required for temperature, shrinkage, etc.

5.3.4 Conditions Always Requiring Inspection

As required in Section 3.5.5, conduct a detailed visual inspection for each of the following conditions:

- Columns with $s/d > 0.5$ and $P/A_g f'_{ce} > 0.3$,
- Shear-controlled walls with $P/A_g f'_{ce} > 0.15$ and $V_{ne}/V_{@Mne} < 0.5$,
- Flexure-controlled walls with $V_{ne}/V_{@Mne} \geq 1.15$ and values of $l_w c/b^2$ exceeding 70, and
- Slab-column connections with gravity shear ratio exceeding 0.4.

In these conditions, s = spacing of shear reinforcement; d = depth of cross-section, distance from extreme compression fiber to centroid of tension reinforcement; P = expected compressive axial load; A_g = cross-sectional area of column; f'_{ce} = expected compressive strength of concrete; V_{ne} = nominal shear capacity per ACI 318, using expected material properties; $V_{@Mne}$ = shear demand at expected flexural capacity, using expected material properties (for walls); and $l_w c/b^2$ = slenderness parameter for walls.

Commentary: Components exhibiting the above conditions are vulnerable to sudden failure with limited warning during strong ground shaking. The behavior of such components can change rapidly with the potential for sudden loss of gravity-load-carrying capacity; therefore, it is not advisable to rely solely on analysis results to determine if such components need to be inspected. Careful inspection and consideration of Visual Damage States defined in

Section 5.5.2 is particularly important for columns and walls with high axial load where cracks are likely to have closed under gravity loads.

5.4 Performance-Critical Force and Deformation Limits

This section provides Performance-Critical Limits for concrete components. Such limits are used to determine Inspection Indicators (Section 3.5.2), reconcile observations from inspection and structural analysis (Section 3.7), and, if necessary, to assist in determining damage classifications (Section 2.2.4.2; Section 4.3).

5.4.1 Classification of Elements, Components, and Actions

Classify structural elements as moment frames, concrete frames with infills, structural walls, braced frames, diaphragms, and foundations consistent with the classifications of Chapter 10 of ASCE/SEI 41.

For each element, identify the constituent components consistent with Chapter 10 of ASCE/SEI 41.

Classify actions in the components as being either deformation controlled or force controlled, consistent with Chapter 10 of ASCE/SEI 41.

Commentary: *ASCE/SEI 41 classifies actions as force or deformation controlled. For concrete components, actions are considered deformation controlled where the component behavior is well documented by test results and deformation modelling parameters and acceptance criteria are provided in Chapter 10 of ASCE/SEI 41. All other actions are considered as force controlled.*

These Guidelines *set Performance-Critical Limits for force-controlled actions based on the expected strength (Sections 5.4.2 and 5.4.4.2) and for deformation-controlled actions based on deformation at initiation of component strength loss (Sections 5.4.3 and 5.4.4.1).*

5.4.2 Performance-Critical Force-Controlled Limits

Calculate strengths used for force-controlled actions using procedures of Chapter 10 of ASCE/SEI 41 except strengths shall be taken equal to expected strengths, Q_{CE}, obtained experimentally or calculated using accepted principles of mechanics and expected material strengths, with strength reduction factor, $\phi = 1.0$.

Unless alternative procedures are provided in Section 5.4.4, define the performance-critical force-controlled limits as being equal to expected strengths, Q_{CE}.

Commentary: *Force-controlled actions experience limited or no ductility prior to degradation of resistance; hence, for such components, the initiation of component strength loss (i.e., DS2) is defined based on a strength rather than a deformation limit.*

These Guidelines *use expected material strengths and $\phi = 1.0$ such that the resulting strength for force-controlled actions is the expected strength, Q_{CE}. This approach is deemed appropriate for the purpose of determining inspection locations based on calculated earthquake demands in accordance with Section 3.5.5 because conservatism is already built into the inspection factor defined in Section 3.5.5.*

This approach is in contrast with the lower-bound strength philosophy adopted by ASCE/SEI 41 for determination of strength of force-controlled actions for assessment or retrofitting of a building.

5.4.3 Performance-Critical Deformation-Controlled Limits

Unless alternative procedures are provided in Section 5.4.4, define the performance-critical deformation-controlled limits as being equal to $\eta = 0.75$ times the value of the deformation *a* or *d*, as applicable, of the generalized force-deformation relationship of Chapter 10 of ASCE/SEI 41.

In addition to the requirement of this section, assess all flexure-controlled components for low-cycle fatigue damage in accordance with Section 5.6. Flexure-controlled components are those whose inelastic deformations occur primarily in flexure without being prematurely curtailed by shear, anchorage, or other relatively low ductility action.

Commentary: *These* Guidelines *define a Performance-Critical Limit for deformation-controlled actions that is equal to a multiplier, η, times the deformation parameter* a *or* d*, whichever is used for the component generalized load-deformation curve in ASCE/SEI 41. Figure 5-4 illustrates the concept. The broken curve defined by points A through E is the generalized load-deformation relationship from ASCE/SEI 41. The deformation at point C is measured by either parameter a or d depending on the nature of the individual component being represented. Point C is defined in ASCE/SEI 41 as the point at which the component resistance decays by 20% from the peak resistance due to damage in the component. These* Guidelines *define the performance-critical deformation limit for deformation-controlled actions as the deformation at initiation of component strength loss (i.e., DS2), which occurs at some deformation less than the deformation at 20% decay in resistance. The deformation defining DS2 is given by either $\eta \times$ a or $\eta \times$ d.*

Values of a *or* d *are obtained from ASCE/SEI 41, except where alternative values are provided in Section 5.4.4 of these* Guidelines*.*

Values of η reported in Table 5-3 were determined from the study of databases of laboratory test data for walls, coupling beams, beams, columns, beam-column joints, and slab-column connections. These studies quantified the relationship between median deformation at initiation of component strength loss (see DS2 in Figure 5-4) and the deformation at point C given by ASCE/SEI 41 (see point C in Figure 5-4). For other components, the results from the study of databases, combined with engineering judgment, were used to arrive at the default value of $\eta = 0.75$ in this section. Where values differ from 0.75, this is due to a number of *factors, including inconsistent levels of conservatism used in developing the ASCE/SEI 41* a *and* d *factors.*

When using linear analysis, parameter η *should be used to adjust m-factors as indicated in Section 3.5.2.*

The fatigue check of Section 5.6 is only required for flexure-controlled components as other response actions do not repeatedly strain the reinforcement as is required to initiate a low-cycle fatigue fracture.

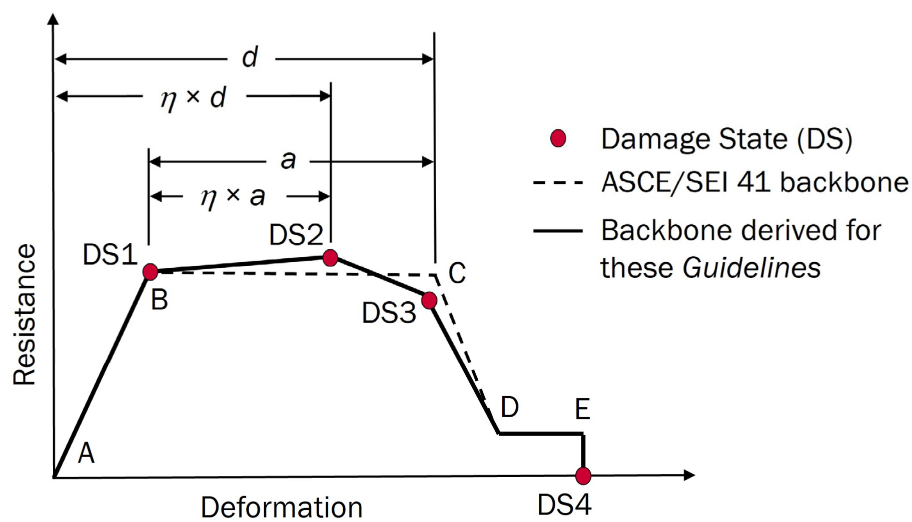


Figure 5-4 Generalized force-deformation relationship for concrete components.

5.4.4 Exceptions to ASCE/SEI 41 Force- or Deformation-Controlled Limits

5.4.4.1 EXCEPTIONS TO PERFORMANCE-CRITICAL DEFORMATION-CONTROLLED LIMITS

Table 5-3 contains exceptions to the default deformation-controlled limits from Section 5.4.3 for specific component types. The performance-critical deformation-controlled limit is taken equal to either $\eta \times a$ or $\eta \times d$, whichever is appropriate, where values of η are in Table 5-3. Values of *a* or *d* are from ASCE/SEI 41 except where “See 5.4.4.2(x)” is indicated in the “ASCE/SEI 41 Modeling Parameter” column of Table 5-3.

Table 5-3 also identifies specific components to be considered as force controlled using the provisions of Section 5.4.2.

Table 5-3 Multiplier on ASCE/SEI 41 Modeling Parameters to Determine Performance-Critical Limits [1]

Element	Component	Classification[2]	ASCE/SEI 41 Modeling Parameter	Multiplier η
Structural wall	Cantilever wall or wall pier	Conforming flexure controlled[3]	*d*	0.80
		Nonconforming flexure controlled[3]	*d*	0.70
		Shear controlled	*d*	0.85
		Shear-friction controlled	*a*	0.60
	Conventionally reinforced spandrel/ coupling beam	Conforming[4]	See 5.4.4.2(a)	0.75
	Diagonally reinforced coupling beams		See 5.4.4.2(b)	0.70
Beam-column frame	Columns	Conforming[5]	See 5.4.4.2(c)	0.75
		Shear controlled[5]	Treat as force controlled (see Section 5.4.2)	
		Splice controlled[5]	Treat as force controlled (see Section 5.4.2)	
		All other columns[5]	*a*	0.50
	Beam-column joints		See 5.4.4.3	
Slab-column frame	Non-prestressed		*a*	0.85
	Post-tensioned		*a*	0.72

(1) For elements and components not covered by this table, use a multiplier of $\eta = 0.75$

(2) Classification in accordance with ASCE/SEI 41-23 unless noted otherwise

(3) Flexure-controlled, conforming walls satisfy the requirements of ACI 318-19 except *A(sh, provided)/A(sh, required)* ≥ 0.7 (instead of ≥ 1.0 in ACI 318-19), $s/d_b \leq 8.0$ (instead of 6.0 in ACI 318-19), and with or without overlapping hoops.

(4) Conforming conventionally reinforced coupling beams are defined as beams with transverse reinforcement consisting of closed stirrups over the entire length of the coupling beam at spacing $\leq d/3$, and V_n greater than that required to develop M_{ne}.

(5) Conforming columns are defined as columns not controlled by splices and with $V_p/V_0 \leq 0.6$, $s/d \leq 0.5$ and $\rho_t \geq 0.002$. Shear-controlled columns are defined as columns not controlled by splices and with $V_p/V_0 > 1.0$. Splice-controlled columns are defined as per ASCE/SEI 41-23.

Commentary: The recommended values for η in Table 5-3 were developed by analyzing results from laboratory test databases for the specific components listed (See Appendix A for more details).

5.4.4.2 EXCEPTIONS TO *a* AND *d* VALUES OF ASCE/SEI 41

Table 5-3 identifies exceptions to the ASCE/SEI 41 modeling parameters *a* and *d*, using a pointer to this section. These exceptions are listed below.

Commentary: This section provides alternative modeling parameters for coupling beams and conforming ductile columns since those specified in ASCE/SEI 41-23 were found to provide biased estimates of deformation capacity based on an analysis of laboratory test databases. Future versions of ASCE/SEI 41 may provide improved modeling parameters that could replace those given below.

5.4.4.2(a): For conventionally reinforced spandrel/coupling beams with conforming transverse reinforcement, the modeling parameter *d* shall be given by:

For $l_n/h \le 1.0$, $d = 0.027$
$l_n/h \ge 3.5$ $d = 0.040$
Linear interpolation is permitted for $1.0 < l_n/h < 3.5$.

5.4.4.2(b): For diagonally reinforced spandrel/coupling beams, the modeling parameter *d* shall be given by:

For $l_n/h \le 0.5$ $d = 0.027$
$l_n/h \ge 3.5$ $d = 0.080$
Linear interpolation is permitted for $0.5 < l_n/h < 3.5$.

5.4.4.2(c): For columns with conforming transverse reinforcement, the modeling parameter *a* shall be given by:

$$a = 0.075 - \frac{P}{A_g f'_{ce}} \qquad (5\text{-}1)$$

where P, A_g, and f'_{ce} were defined in Section 5.3.4 and *a* must be greater than or equal to 0.03 and less than or equal to 0.06.

5.4.4.3 EXCEPTIONS TO FORCE- AND DEFORMATION-CONTROLLED PROCEDURES FOR BEAM-COLUMN JOINTS

Classify beam-column joints according to the joint failure mode. To determine the joint failure mode, compare the joint horizontal shear demand, V_{uj}, with the joint shear strength, V_{nj}, as shown in Figure 5-5. V_{uj} is calculated from expected loads and nominal sectional strengths determined based on expected material strengths with $\phi = 1.0$, and can be limited by either beam or column moment strength or shear strength. V_{nj} is calculated in accordance with ASCE/SEI 41 except using expected material strengths with $\phi = 1.0$.

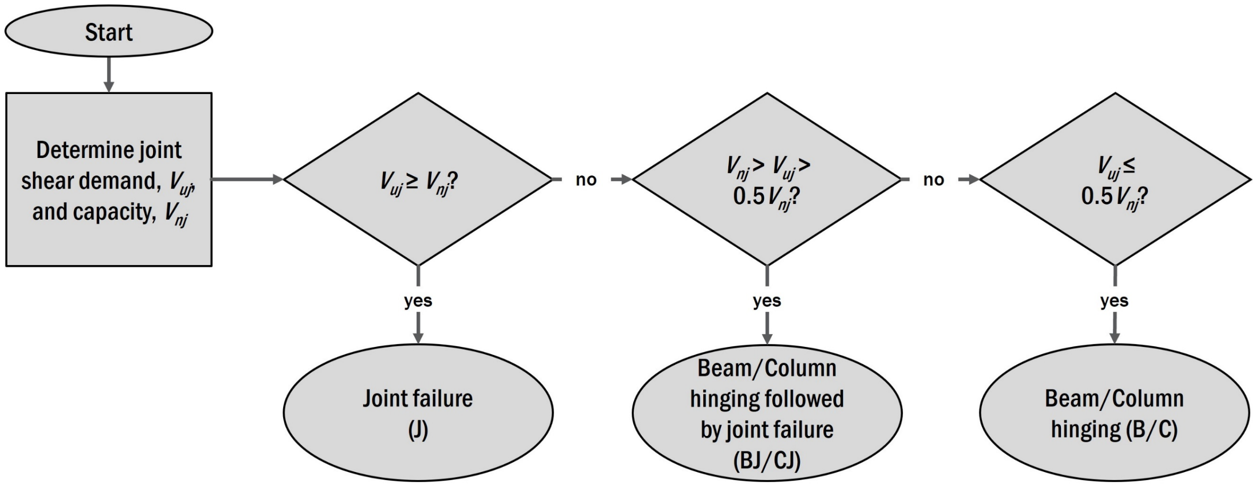


Figure 5-5 Classification of frame assembly mechanisms and joint failure mode.

The resulting failure mode is one of the following:

- J Failure: Failure occurs in the joint prior to moment or shear failure of the beams or columns framing into the joint.
- BJ or CJ Failure: Failure occurs in the joint after flexural hinging in the beam or column.
- B or C Failure: Failure occurs by flexural hinging in the beam or column without joint failure.

Where beam or column strength is limited by beam or column shear failure, rather than flexural failure, that shear failure may occur before joint shear failure (not shown in Figure 5-5).

Joints classified as J failure are treated as force controlled and assessed using Section 5.4.2.

Joints assessed as B or C are expected to be limited by either beam or column deformation or strength capacities, and the associated beam-column joints are expected to not reach Performance-Critical Damage states. Inspection therefore focuses on the beam or column and the joint does not need inspection.

Where analysis in accordance with this section indicates BJ failure, and the structural analysis in accordance with Chapter 3 identifies an inspection location in the associated beams, the inspection should include the beam-column joint. Likewise, where analysis in accordance with this section indicates CJ failure, and the structural analysis in accordance Chapter 3 identifies an inspection location in the associated columns, the inspection should include the beam-column joint.

> ***Commentary:*** *Classification of joints is based on a mechanism analysis of the beam-column-joint subassembly. To determine joint shear demand, each joint is analyzed to assess*

whether the demands are likely to be limited by either beam strength or column strength (Figure 5-6). Beam or column strength can be limited by either moment strength or shear strength. The smallest strengths of the members framing into the joints determines the horizontal joint shear force, V_{uj}. *ACI 352R-02,* Recommendations for Design of Beam-Column Connections in Monolithic Reinforced Concrete Structures *(ACI, 2002), describes an approach for calculating the horizontal joint shear force in cases where strength is limited by moment capacity of either the beams or columns.*

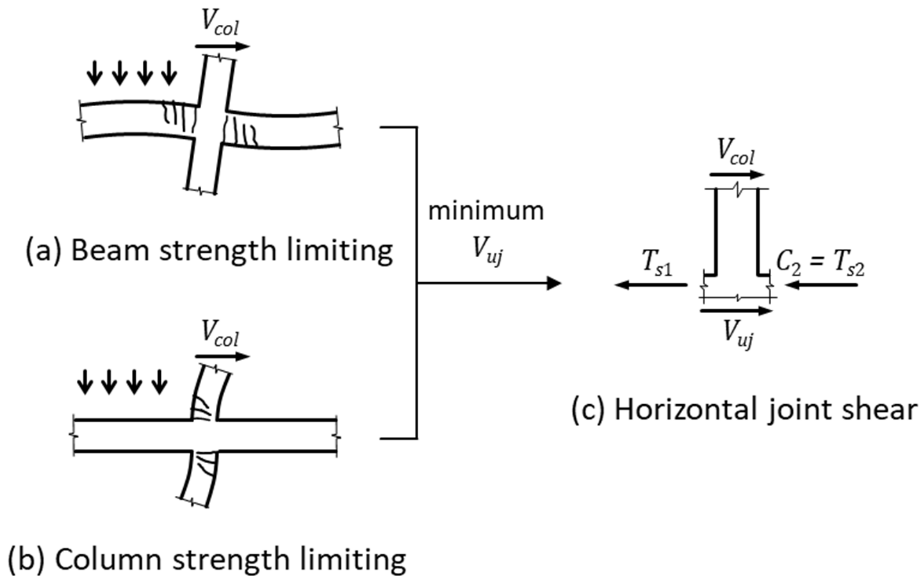


Figure 5-6 Calculation of joint horizontal shear demands.

Laboratory tests demonstrate that joint shear strength decays as flexural yielding in adjacent beams and columns penetrates into the joint (Figure 5-7). Consequently, inelastic response that starts with either beam or column flexural yielding can transition into BJ or CJ joint failure as the joint strength degrades to the joint demand associated with flexural yielding. Joint strength degradation does not fall below approximately $0.5V_{nj}$. *Thus, beams or columns that produce joint shear demands below this lower limit are not expected to sustain joint shear failure.*

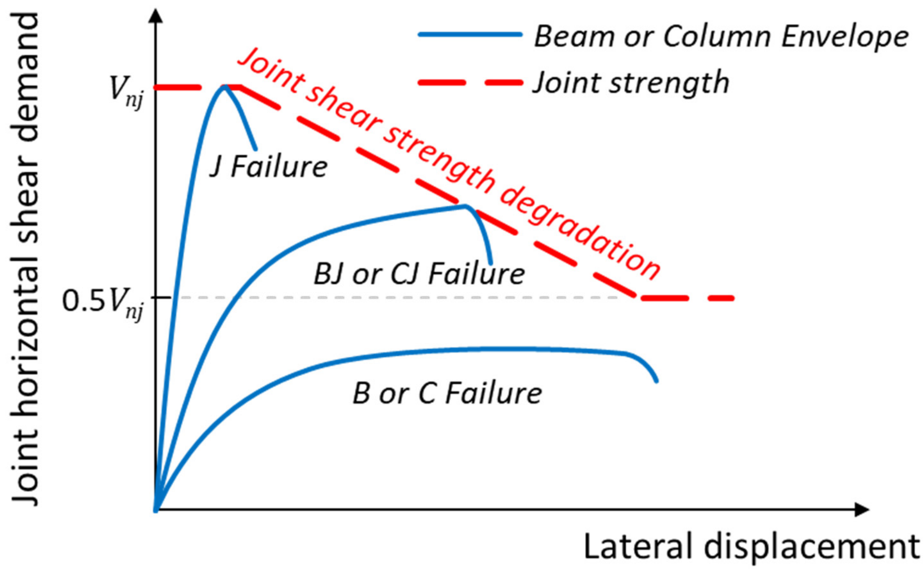


Figure 5-7 Classification of frame subassembly mechanisms.

5.5 Classification of Damage

As described in Section 2.2.4 and Section 4.3, these *Guidelines* use the damage observed during inspection to determine if any components are in DC2, and hence, require performance-critical repair. Section 5.5.1 describes key damage characteristics associated with DC2 for concrete components. In some cases, the observed damage can be classified as DC2 based simply on consideration of these key damage characteristics, without the need to consult the VDS databases.

The VDS databases were developed to help the engineer further confirm and possibly refine the classification of the damage. The VDS databases, described in Section 5.5.2 and presented in detail in Appendix B, can be used to further refine the classification of damage observed for each component as being DC0, DC1, or DC2.

As described in Section 5.5.3, analysis results are only used to classify damage if consideration of the observed damage in Section 5.5.1 and Section 5.5.2 are inconclusive.

5.5.1 Key Damage Characteristics Associated with DC2

This section provides descriptions of key damage characteristics associated with DC2 for concrete components. If damage is classified as DC2, performance-critical repair will be required. For concrete buildings, any of the following damage characteristics may indicate the damage can be classified as DC2:

- Core crushing, that is, crushing that extends beyond cover delamination or cover spalling and into the core (Figure 5-8),
- Bar buckling, with spalling, possibly crushing extending into the core (Figure 5-9),

- Splice failure or severe splitting cracks along longitudinal reinforcement (Figure 5-10),
- Wide (>2 mm, or about 1/16 in.) inclined cracks indicative of shear failures versus relatively minor or hairline cracks associated with concrete cracking (Figure 5-11),
- Wide (>2 mm, or about 1/16 in.) inclined cracks or severe concrete spalling indicative of beam-column joint shear or anchorage failure (Figure 5-12),
- Flexural or punching shear damage at slab-column connections (Figure 5-13),
- Damage to floor diaphragms at points of high stress (e.g., chords, collectors, precast panel connectors or supports, reentrant corners) impacting load path (Figure 5-14), or
- Damage to foundations such as punching failures at columns or at wall boundaries, inclined cracks indicative of shear failure, and fracture of the footing-pile interfaces.

> ***Commentary:*** *Figure 5-8 through Figure 5-14 are provided to illustrate different types of damage that might be anticipated during an inspection. The damage photographs and DC2 damage characteristics described in this section may be sufficient for an engineer to classify observed damage as DC2. If this classification is not evident from the generic descriptions in this section, the more detailed VDS guidance in Section 5.5.2 should be used.*
>
> *Other post-earthquake assessment guidelines (e.g., FEMA 306) adopt residual crack width as a metric to evaluate the residual capacity of concrete structures. However, there are insufficient residual crack width data from experimental tests to develop and validate an approach based on residual crack width to assess the residual capacity and repairability of earthquake-damaged buildings. Additionally, crack widths in concrete components following an earthquake can be influenced by a number of factors (e.g., displacement history, loading rate, axial load) and might not be a reliable indicator of the demands experienced by the concrete components. Hence, residual crack width data are generally not a sufficient standalone metric to classify the damage of a structural component. The use of residual inclined crack width, however, may provide some useful information for shear-controlled components. Appendix B uses data from laboratory tests to select a residual inclined crack width limit for shear-controlled components. Values are in the range of 1.5 mm to 2.0 mm (about 1/16 in.); however, due to the limited data and possible inaccuracies in crack width measurement (and to promote use of other factors), a common limit of 2 mm (about 1/16 in.) is recommended for all components.*
>
> *Surface spalls might include loss of concrete cover (Figure 5-8a) or delamination between core and cover concrete that is not visually apparent but can be identified using sounding methods (delaminated regions will sound hollow when tapped with a solid object). Spalling with evidence of bar buckling might appear as local cracking and minor concrete spalling at component corners (e.g., wall or column edge/corner; Figure 5-9). Significant cover spalling may occur prior to core crushing in ductile (conforming) components, but is not indicative of*

DC2 on its own. Core crushing is commonly accompanied by bar buckling and possibly fracture and/or hook opening of transverse reinforcement (Figure 5-8b). Spalling without bar buckling/fracture and/or core crushing is typically not considered DC2.

Bond or splice failures are often evident in splitting cracks along the length of the reinforcement and, as the failure progresses, spalling of cover concrete exposing the reinforcement. Figure 5-10a shows a splitting crack along the longitudinal reinforcement of a column. Figure 5-10b shows spalled cover concrete along a splice in a wall boundary element.

Hairline cracks in shear-controlled components (Figure 5-11a) are likely to occur if the component is loaded beyond concrete cracking, but reinforcement providing shear resistance has not yet reached yield. This condition normally does not constitute a performance-critical repair condition. Once yielding of reinforcement occurs, cracks open and sliding along an inclined crack may be observed just prior to shear and possibly axial failure (Figure 5-11b and 5-11c). As described in Appendix B, a 2 mm (about 1/16 in.) crack width can be used as a good indication of transverse reinforcement yielding. In such conditions, core concrete has likely degraded due to abrasion along inclined cracks (Wight and Sozen, 1973).

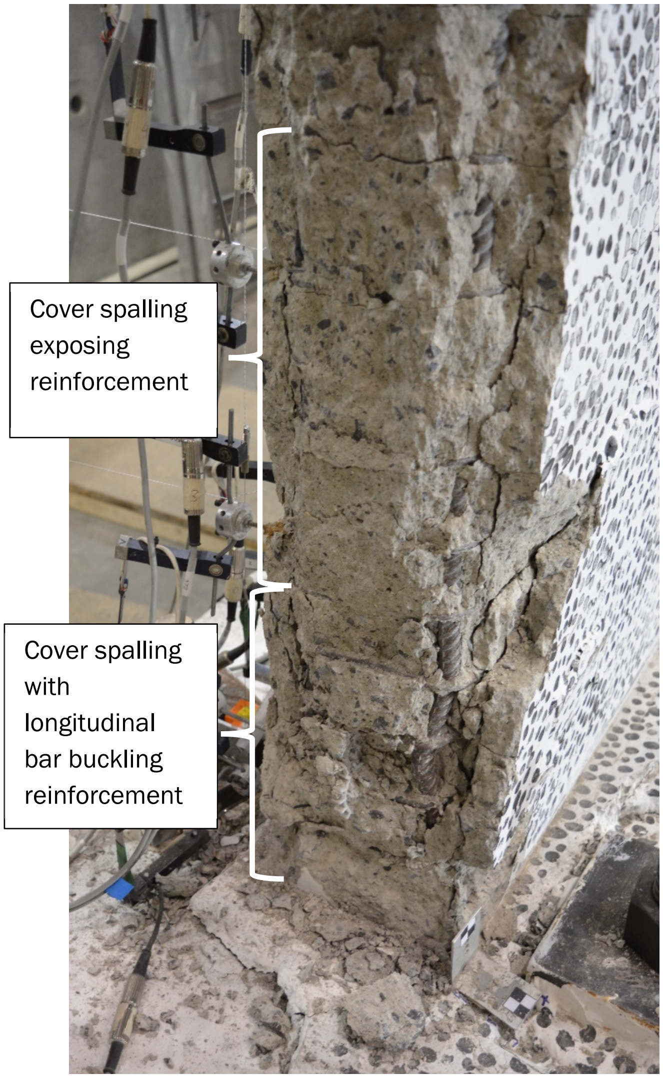
Damage to beam-column joints may present as hairline diagonal cracks, possibly accompanied by minor cover spalling (Figure 5-12a). Joint failure is commonly apparent in wide diagonal cracks, possibly with anchorage failure or buckling of reinforcement entering the joint and possibly with vertical and/or lateral movement of the members framing into the joint (Figure 5-12b).

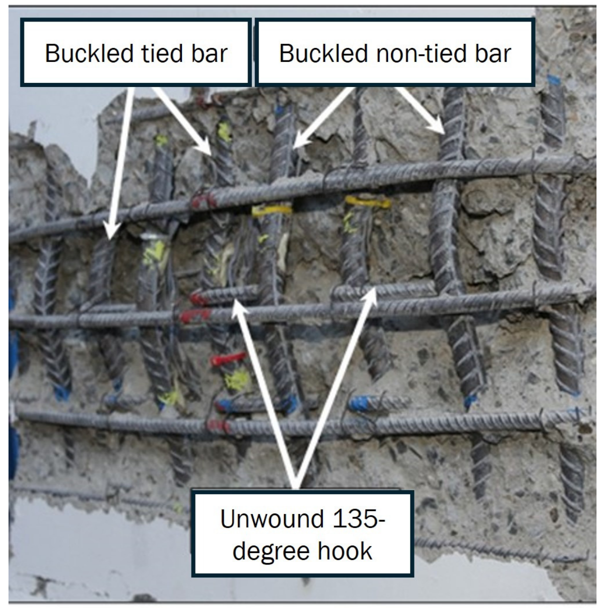
Damage to slab-column connections may be apparent in flexural cracks extending across the width of a slab panel, which would not normally require performance-critical repair. Large flexural rotations may result in reinforcement buckling, which would require repair. Damage may also occur in the form of punching shear failure in which a failure surface extending around the column projects through the slab, often with a visible vertical offset and spalling of the top surface of the slab at a distance about two slab thicknesses away from the face of the column (Figure 5-13).

Diaphragm damage in buildings with diaphragms may present itself as wide cracks around the perimeter of the diaphragm, which are mainly attributable to beam elongation due to flexural yielding (Figure 5-14a). Fracture of collector bars may be evident especially where the diaphragm connects to wall boundaries (Figure 5-14b).

(a) (b)

Figure 5-8 **Examples of concrete core damage: (a) cover spalling and hairline cracking that may expose the reinforcement cage but not extend within the boundaries of the reinforcement cage, not indicative of Performance-Critical Damage (from 2007 Pisco, Peru Earthquake; photo credit: S. Pujol); (b) core crushing that extends within the boundary of the reinforcement cage (from 2016 Meinong, Taiwan Earthquake; photo credit: of S. Pujol).**





(a) (b)

Figure 5-9 Examples of bar buckling damage: (a) cover spalling with bar buckling (photo credit: A. Shegay); (b) cover spalling, core crushing, and bar buckling (photo credit: C. Arteta).

(a) (b)

Figure 5-10 Examples of lap splice damage: (a) splitting cracks along longitudinal reinforcement, indicative of bond distress and possible lap-splice distress (from 2017 Mexico Earthquake; photo credit: S. Pujol); (b) wall boundary element lap splice failure (from 2010 Maule, Chile Earthquake; photo credit: S. Pujol). (Note: in the case of (a), limited intrusive inspection (chipping cover) to expose lap splice would be useful for determining damage classification).

(a)

(b)

(c)

Figure 5-11 Examples of concrete cracking and shear failures: (a) hairline diagonal cracking, not indicative of Performance-Critical Damage (from 2023 Turkey Earthquakes; photo credit: S. Pujol); (b) diagonal cracking indicative of column shear failure (from 1999 Izmit, Turkey Earthquake; photo credit: National Information Service for Earthquake Engineering); (c) structural wall cracking indicative of shear failure (from 1999 Duzce, Turkey Earthquake; photo credit: S. Pujol).

(a)

(b)

Figure 5-12 Examples of beam-column joint damage: (a) joint cracking and cover spalling indicative of minor shear "working" of joint, not indicative of Performance-Critical Damage (photo credit: E-Defense); (b) joint failure including shear and crushing damage of joint core (from 1999 Izmit, Turkey Earthquake; photo credit: National Information Service for Earthquake Engineering).

(a) (b)

Figure 5-13 Examples of reinforced concrete slab damage: (a) flexure or flexure-shear failure at perimeter of drop capital (photo credit: T. Sabol); (b) plan view of punching shear failure (photo credits: S.R. Sanchez).

(a) (b)

Figure 5-14 Examples of reinforced concrete diaphragm damage: (a) diaphragm damage due to beam elongation (photo credit: D. Bull); (b) fracture of collector (photo credit: EERI, 1996).

5.5.2 Visual Damage States

The VDS databases (Rodriguez Sanchez et al., 2024), which can be freely accessed through the data repository provided by NHERI DesignSafe, include photographs of concrete components from laboratory tests corresponding approximately to states DS1, DS2, DS3, and DS4 (Figure 5-15). For some databases, an elastic Damage State (DS0) is also provided and, for others, an intermediate Damage State between DS1 and DS2 (namely DS1.b) is also included.

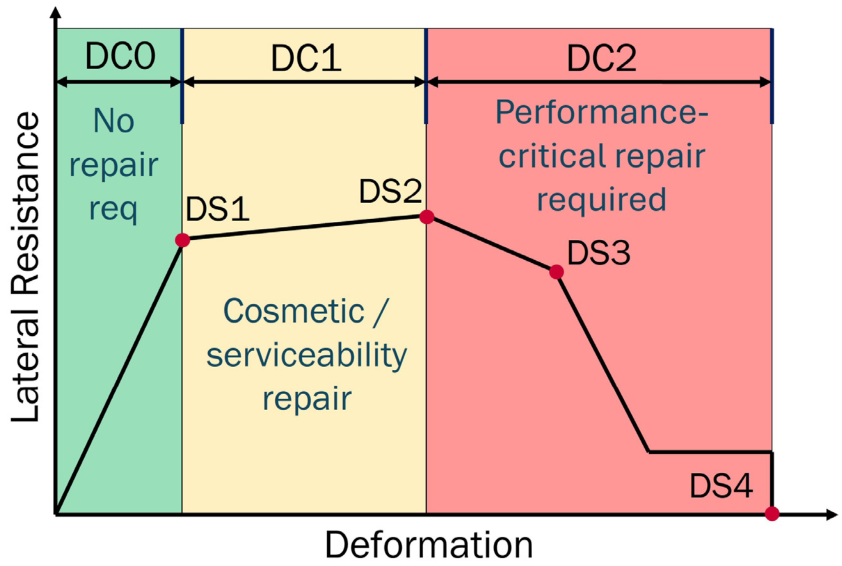


Figure 5-15 Damage States (DS1 through DS4) and Damage Classes (DC0 through DC2) in relation to component backbone response for deformation-controlled actions.

Photographs in the VDS databases can be helpful in differentiating Damage States likely to be encountered. They also can be used as an aid in classifying observed damage in DC0, DC1, and DC2.

Table 5-4 lists the elements, components, and classifications for which VDS databases were developed. See Appendix B for more details. VDS databases were not developed for all elements, components, and actions that an engineer might encounter in an earthquake-damaged building. Where specific databases are not available for a particular condition, use the information in Appendix B and the critical damage modes in Section 5.5.1 for general guidance on classifying observed damage.

Table 5-4 Components and Actions for Which Visual Damage States are Provided

No.	Element	Component	Description
1	Walls	Cantilever walls or piers	Conforming flexure controlled
2			Nonconforming flexure controlled
3			Lap-splice controlled
4			Shear controlled: Flexure-shear controlled walls and diagonal tension/compression-controlled walls
5			Shear-friction controlled
6		Coupling beams	Diagonally reinforced and conventionally reinforced

Table 5-4 Components and Actions for Which Visual Damage States are Provided (cont.)

No.	Element	Component	Description
7	Slab-column frames	Slab-column connections	Punching-shear controlled
8	Beam-column frames	Columns	Nonconforming shear controlled
9			Nonconforming flexure-shear controlled
10			Conforming ductile
11			Lap-splice controlled
12		Beams	Conforming ductile
13		Beam-column joints: Interior	Beam-yielding/joint-failure controlled
14		Beam-column joints: Exterior	Joint-failure controlled

When using the VDS databases, it will be necessary to determine the appropriate database for the condition under consideration. Table 5-5, repeated from Table B-2 in Appendix B, provides the criteria for classification of various concrete components and the corresponding VDS database.

Table 5-5 Criteria for Determining the Appropriate VDS Database

Component Type	Criterion 1	Criterion 2	Database
Walls	$V_{ne}/V_{@Mne} > 1.15$ and $V_{nfe} > V_{@Mne}$	$s/d_b \leq 8.0$, and $A_{sh,provided}/A_{sh,required} \geq 0.7$	1. Conforming Flexure-Controlled Walls
		$s/d_b > 8.0$, and/or $A_{sh,provided}/A_{sh,required} < 0.7$	2. Nonconforming Flexure-Controlled Walls
		$l_s/l_d < 1.625$	3. Lap-Splice-Controlled Walls
	$V_{ne}/V_{@Mne} \leq 1.15$ and/or $V_{nfe} \leq V_{@Mne}$	$V_{ne} < V_{nfe}$	4. Shear-Controlled Walls
		$V_{ne} \geq V_{nfe}$	5. Shear-Friction-Controlled Walls

Table 5-5 Criteria for Determining the Appropriate VDS Database (cont.)

Component Type	Criterion 1	Criterion 2	Database
Columns	$V_p/V_o > 1.0$	N/A	8. Nonconforming Shear-Controlled Columns
	$0.6 < V_p/V_o \leq 1.0$	N/A	9. Nonconforming Flexure-Shear-Controlled Columns
	$V_p/V_o \leq 0.6$	ACI 318 conforming seismic details	10. Conforming Ductile Columns
	$l_{splice}/l_d < 1.625$	N/A	11. Lap-Splice-Controlled Columns

where:
V_{ne} = Expected shear capacity per ACI 318, using expected material properties
M_{ne} = Expected flexural capacity per ACI 318, using expected material properties
$V_{@Mne}$ = Shear demand at expected flexural capacity, using expected material properties
V_{nfe} = Expected shear friction capacity per ACI 318, using expected material properties
l_s = Length of splice provided
l_d = Development length per ACI 318, using expected material properties
V_p = Shear demand at expected flexural strength (for columns)
V_o = Expected undegraded shear capacity, according to ASCE/SEI 41 and ACI 369

Commentary: *Appendix B provides detailed guidance on how the VDS databases were developed and how to use the databases to identify the DC of the earthquake-damaged component. The key steps are summarized below:*

1. *<u>Select the appropriate VDS database(s)</u>: Classify the component as summarized in Table 5-5, to identify the appropriate VDS database. If the component is near to the boundary between two databases, the engineer may need to consult two or more VDS databases.*
2. *<u>Develop initial estimate of DC</u>: Review the general description of the Damage States (DS1 through DS4) on the "Guidance" sheet to develop an initial estimate of the likely DC of the earthquake-damaged component.*
3. *<u>Select representative laboratory tests</u>: Tables B-3 through B-6 identify the key parameters to use in the selection of representative laboratory tests. Determine the values of those parameters for the earthquake-damaged component, being sure to consider possible range of some parameters during earthquake shaking (e.g., variable axial load). Filter the database to identify laboratory tests with similar values of the key parameters. If an exact match cannot be identified (typically the case), two or more tests should be selected. Consult "Guidance on identifying representative tests" on the "Guidance" sheet for further details.*
4. *<u>Determine the DC for the earthquake-damaged component</u>: Compare the observed state of the earthquake-damaged component with the photos provided for the selected laboratory tests. Note that because photos were not always available at the prescribed*

Damage States, in some cases, photos for intermediate Damage States are provided (e.g., DS1.b). Start with the photos representing the initial estimate of DC from Step 2. Do these photos match the extent of damage observed in the earthquake-damaged component? Pay particular attention to the description of damage accompanying the photos in the VDS database. It should be emphasized that, for the purpose of determining whether Performance-Critical Damage has occurred, it typically is only required to determine if the component is in DC2 (i.e., past DS2). To aid in this determination, in some cases the databases also provide a fraction of DS2 associated with each photo (e.g., 0.5DS2 indicates the photo was taken at a deformation demand half of that at DS2). The outcome of this process is the DC for the earthquake-damaged component. Consult the "Guidance" sheet for further details.

Considerable judgement may be required in interpreting the relationship between the observed damage in the field and the images from laboratory tests. Consulting the general descriptions of severe damage in Section 5.5.1 will also assist in arriving at the decision regarding the likely DC.

5.5.3 Use of Structural Analysis to Classify Damage

If VDS comparison described is Section 5.5.2 is inconclusive in determining if the component is in DC2 or not, the Performance-Critical Limits (Section 5.4) can be used to inform the identification of the appropriate DC.

Commentary: These Guidelines *use the damage observed during inspection to determine if any components are in DC2, and hence, require performance-critical repair. Results of analysis should only be used if the engineer is unsure if the component is in DC2 after following the guidance of Sections 5.5.1 and 5.5.2 (i.e., observed damage is close to that described as DS2, but it is not clear which side of this key Damage State the component sits).*

5.6 Low-Cycle Fatigue Damage Assessment

5.6.1 Fatigue Categories and Required Actions

Fatigue damage assessment is only required for components assigned to damage class DC1 and DC2, as determined from Section 5.5. It shall be assumed for components assigned to damage class DC0 that reinforcement is undamaged.

Conduct a preliminary inspection in accordance with Section 3.3 or a detailed visual inspection in accordance with Section 3.6.1 with particular attention focused on the condition of the reinforcing bars and whether spalling "sufficiently exposes" any bars to an extent that may suggest buckling may have occurred during the damaging earthquake. In determining the Fatigue Category, a bar is deemed to be "sufficiently exposed" by cover spalling if the bar is exposed around 50% or more of its circumference for a continuous length of four times the bar diameter ($4d_b$) or more. A bar with less exposure is assumed in these *Guidelines* not to have experienced buckling during the damaging earthquake.

Based on the inspection, assign reinforcing bars to one of four fatigue damage categories:

- Fatigue Category 1: Cover not spalled to sufficiently expose bar (as defined above). Reinforcement satisfies ACI 318 minimum reinforcement ratio requirement.
- Fatigue Category 2: Cover not spalled to sufficiently expose bar. Longitudinal reinforcement less than ACI 318 minimum requirement.
- Fatigue Category 3: Cover spalled to sufficiently expose bar. Reinforcement buckling or fracture not visible.
- Fatigue Category 4: Reinforcement buckling, other residual distortion, or fracture visible.

For Fatigue Category 1, Performance-Critical Damage is not indicated, and repair for low-cycle fatigue is not required.

For Fatigue Categories 2 and 3, the reinforcement shall be further assessed according to Section 5.6.2 to determine if the remaining fatigue life of the reinforcement is sufficient. If the remaining fatigue life is deemed insufficient, repair of Performance-Critical Damage is required to address the reduced strain capacity of the reinforcement due to fatigue damage.

For Fatigue Category 4, repair of Performance-Critical Damage is required to address the bar fracture and/or reduced strain capacity of the reinforcement due to fatigue damage.

> *Commentary: Bar buckling or low-cycle fatigue (LCF) of reinforcement may lead to fracture of reinforcement. Consequently, consideration is required of the possibility that LCF has materially compromised the reinforcement strain capacity in an earthquake-damaged building. Figure 5-16 illustrates the fatigue assessment process adopted by these* Guidelines*.*

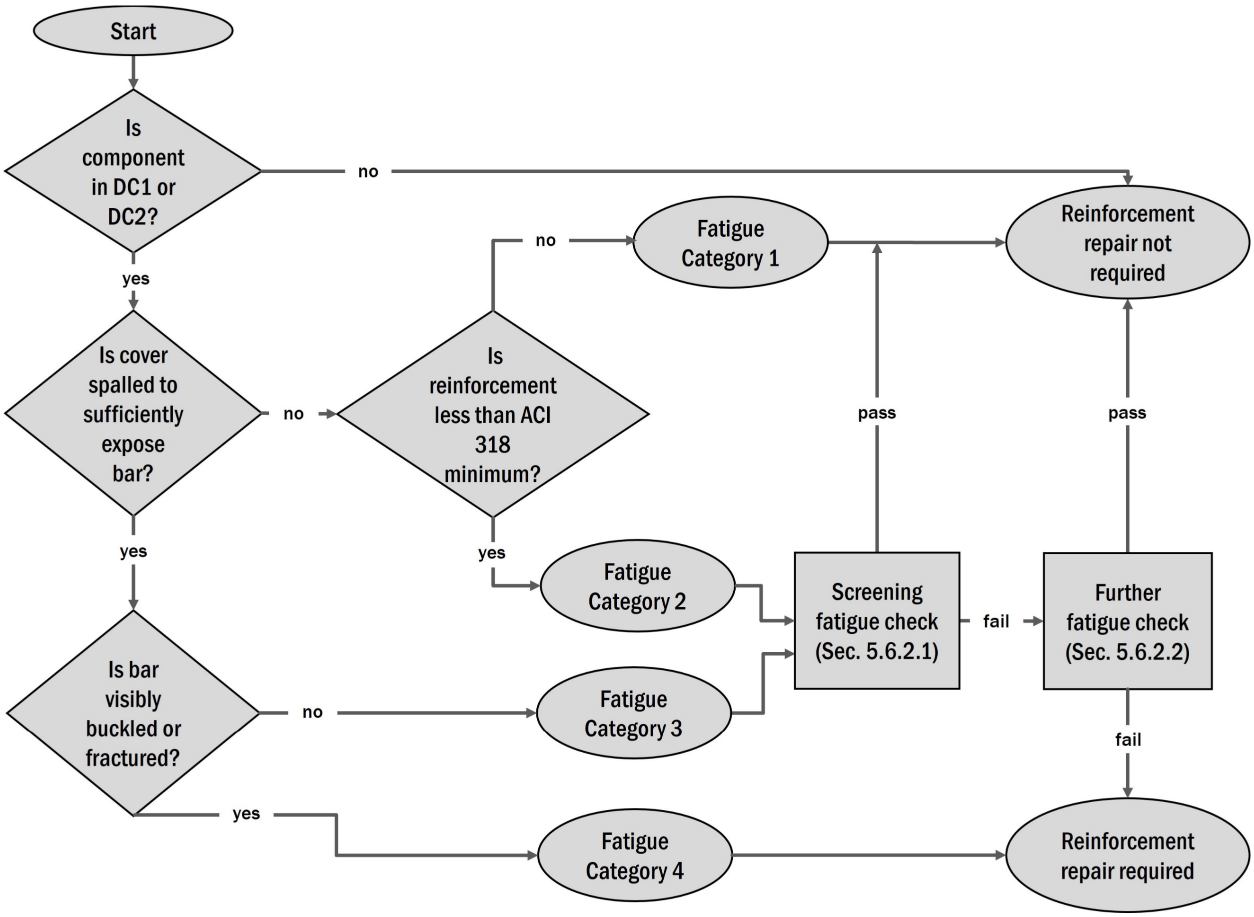


Figure 5-16 Low-cycle fatigue damage assessment process.

Reinforcement in Fatigue Category 1 is likely to have been well supported by cover concrete, such that buckling was unlikely, and reinforcement strains should have been well distributed because the section satisfies ACI 318 minimum reinforcement ratio requirements. This reinforcement is unlikely to have sustained Performance-Critical Damage and does not require performance-critical repair.

Reinforcement in Fatigue Category 2 is unlikely to have buckled, but it may have sustained fatigue damage or even fracture because the section did not satisfy ACI 318 minimum longitudinal reinforcement ratio requirements. Component cross sections that do not satisfy ACI 318 minimum longitudinal reinforcement ratio requirements may have local reinforcement tensile strength less than the concrete cracking strength. This is expected to cause reinforcement strains to concentrate near cracks such that risk of reinforcement damage or fracture is increased. This reinforcement should be checked for fatigue damage via screening (Section 5.6.2.1) or further fatigue checks (Section 5.6.2.2). Intrusive inspection, per Section 3.8 and Section 5.3.3, may be warranted.

Reinforcement in Fatigue Category 3 may have experienced buckling during peak drift despite buckling not being visible during visual inspection. Similar to Fatigue Category 2, this *reinforcement should be further checked for fatigue damage via screening (Section 5.6.2.1) or further fatigue checks (Section 5.6.2.2).*

Reinforcement in Fatigue Category 4 has visibly buckled or fractured. Consequently, the reinforcement requires performance-critical repair without further analysis.

If it is determined from this process that reinforcement repair is required, Section 5.9.5 on repair techniques for steel reinforcement should be consulted.

5.6.2 Low-Cycle Fatigue Assessment of Reinforcement Assigned to Category 2 or Category 3

The residual fatigue life of reinforcement in an earthquake-damaged component can be considered sufficient if either of the following is satisfied:

- The reduction in the fatigue life of the reinforcement due to the damaging earthquake is negligible, or
- The impacted reinforcement is able to withstand the expected demands of a future Risk-Targeted Maximum Considered Earthquake (MCE_R) without fracture.

The fatigue condition of reinforcement may be assessed using either the screening fatigue check described in Section 5.6.2.1 or the further fatigue checks described in Section 5.6.2.2 and Appendix C.

Commentary: *Appendix C provides further guidance on assessment of residual fatigue capacity.*

The criteria to assess LCF in this section are applicable to reinforcement that is not susceptible to strain aging. Reinforcement that contains trace amounts of Vanadium (Erasmus and Pussegoda, 1980; Restrepo-Posada, 1993) and high-strength reinforcing steel (Grade 60 and above) used in the United States are generally considered not to be susceptible to strain aging.

Poorer fatigue performance is expected for reinforcement produced from mild steel (Grade 40), high carbon steel without Vanadium, or high-strength steel produced by cold working, all of which are susceptible to strain aging. The issue of LCF in reinforcement susceptible to strain ageing is a topic of ongoing research (Loporcaro et al., 2018).

5.6.2.1 SCREENING FATIGUE CHECK

Two alternatives are available for the screening fatigue check:

- Consideration of the maximum reinforcement tensile strain estimated to have occurred during the damaging earthquake, or

- Checking of component-specific criteria for beams and walls.

If an element or reinforcing bar satisfies either of the above checks then the reinforcement may be considered to have sufficient residual capacity, i.e. performance-critical damage is not indicated, and repair of reinforcement is not required.

Strain-Based Screening Fatigue Check

The LCF condition of reinforcement can be assessed based on the maximum reinforcement tensile strain estimated to have occurred during the damaging earthquake using the following steps:

1. Estimate the maximum reinforcement tensile strain during the damaging earthquake.
2. Calculate S_{a1} for the damaging earthquake and for the MCE_R at the site.
3. Use the ratio of $S_{a1}/S_{a1,MCER}$ and the significant duration ($D_{5\text{-}95}$) of the damaging earthquake to determine the permitted reinforcement strain from Figure 5-17.
4. Compare the maximum strain from step 1 with the limit from step 3. If the maximum strain estimated in step 1 is less than the limit calculated in step 3, then the strain-based screening fatigue check is satisfied and repair of the reinforcement is not required.

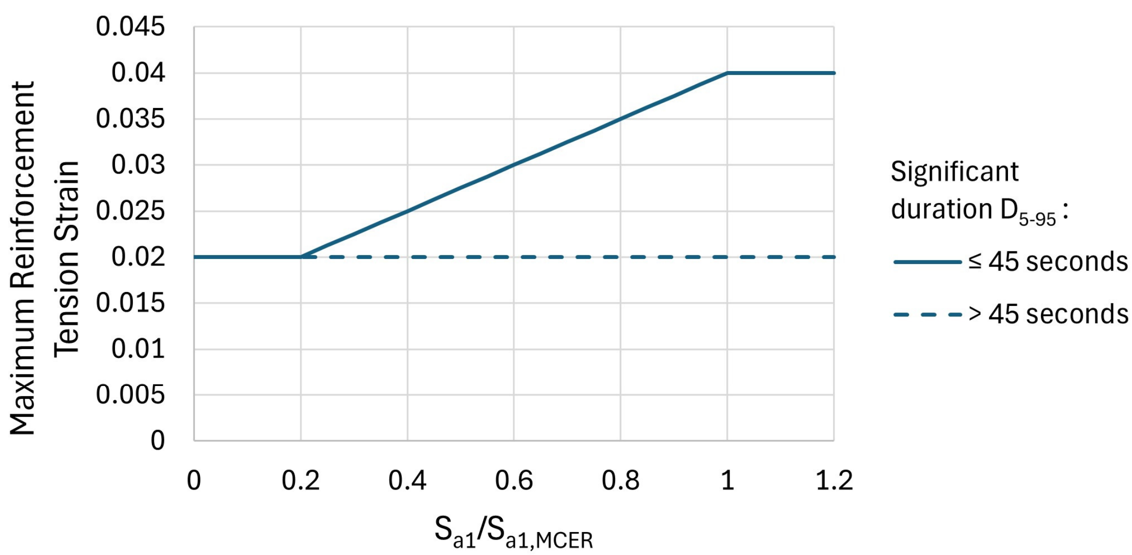


Figure 5-17 Relationship between permitted reinforcement strain calculated for the damaging earthquake and the shaking intensity of the damaging earthquake.

> *Commentary:* *Guidance on how to estimate the maximum reinforcement tensile strain can be found in Appendix C. The number of cycles is an important parameter in determining fatigue capacity of reinforcement. The number of cycles that occur is dependent on the significant duration (*$D_{5\text{-}95}$*) of the damaging earthquake.*

For earthquakes where the significant duration ($D_{5\text{-}95}$*) does not exceed 45 seconds, the maximum permitted reinforcement tensile strain increases as the intensity of the shaking in the damaging earthquake, measured by* S_{a1}*, increases relative to* S_{a1} *for the* MCE_R *at the site. This is because it is acceptable for a larger earthquake to use up a higher proportion of the total fatigue life of the reinforcement. Further detail regarding the basis for the permitted strain limits is provided in Appendix C.*

For longer duration earthquakes, the number of cycles assumed in the calculations used to develop the intensity-dependent strain limit may not be valid. Instead, a flat strain limit of 0.02 is specified for longer earthquakes.

Component-Specific Screening Fatigue Check

As an alternative to the strain-based screening fatigue check above, it is permitted to determine the LCF condition of reinforcement in Fatigue Category 3 for beams and walls using the specific provisions outlined in this section, without calculating the maximum reinforcement tensile strain.

The specific provisions for beams and walls are based on assumed plastic hinge lengths that may not be applicable to under-reinforced elements. Consequently, they are not considered applicable to elements in Fatigue Category 2.

Beams

For reinforced concrete beams, other than diagonally reinforced coupling beams, the residual fatigue life of the reinforcement shall be considered sufficient if all the following conditions are satisfied:

- The maximum chord rotation demand determined in accordance with Section 3.4.3 is less than 0.02 rad, and
- The effective plastic hinge length (as defined in Equation 5-2) of the beam element is greater than 0.4 times the member depth.

$$L_p = k_{lp} L_c + L_{sp} \geq 2L_{sp} \tag{5-2}$$

where:

k_{lp} $= 0.2\left(\dfrac{f_{ue}}{f_{ye}} - 1\right) \leq 0.08$

L_c = shear span, i.e., the distance of the critical section from the point of contraflexure

L_{sp} = strain penetration length $= 0.15 f_{ye} d_b$ where the units of f_{ye} are ksi, or $0.022 f_{ye} d_b$ if the units of f_{ye} are MPa

f_{ye} = expected yield strength of longitudinal reinforcement

d_b = diameter of longitudinal reinforcement

f_{ue} = expected ultimate strength of the longitudinal reinforcement.

Commentary: *If both conditions are satisfied, it is not necessary to estimate the strain demands on the reinforcement during the damaging earthquake.*

The assumptions underpinning this check are not applicable to diagonally reinforced coupling beams.

Walls

For reinforced concrete walls the residual fatigue life of the reinforcement shall be considered sufficient if all the following conditions are satisfied:

- Longitudinal boundary element reinforcement satisfies the minimum quantity required by ACI 318, and
- The maximum displacement at the height of contraflexure determined in accordance with Section 3.4.3 is such that:

$$\Delta \leq \frac{2\varepsilon_y}{l_w}\frac{L_c^2}{3} + 0.01L_c \tag{5-3}$$

where:

L_c = shear span, i.e., the distance of the critical section from the point of contraflexure

ε_y = probable yield strain of longitudinal reinforcement

l_w = length of wall

Commentary: *If both conditions are satisfied, it is not necessary to estimate the strain demands on the reinforcement during the damaging earthquake.*

The displacement limit specified by Equation 5-3 is based on the premise that the magnitude of low cycle fatigue degradation is acceptable provided the plastic rotation imposed on a hinge does not exceed 0.01 radians. This plastic rotation limit is consistent with the lower bound strain limit of 2% in Figure 5-17. Using the strain-based screening approach may provide a less conservative estimate of fatigue capacity.

5.6.2.2 FURTHER DETAILED FATIGUE CHECK

If reinforcement fails the screening checks described in Section 5.6.2.1 (strain-based or component-specific checks), more detailed checks on reinforcement condition can be undertaken.

Further fatigue checks can be undertaken in two ways, namely:

- By a simplified fatigue life assessment using artificial displacement histories to represent the demands previously imposed by the damaging earthquake, and likely to be imposed by an MCE_R, or
- By using a detailed fatigue damage assessment methodology, such as rainflow counting, to determine a fatigue damage summation based on strain demands obtained from response history analysis.

Additional discussion regarding these further fatigue checks can be found in Appendix C.

> *Commentary: The intent of the assessment process described in Figure 5-16 is to limit the number of cases where these further fatigue checks are required to only a limited number of cases. It is anticipated that most concrete components, and indeed most buildings, will not require such checks to be performed.*

5.7 Modeling Parameters for Damaged Components

5.7.1 Effective Stiffness and Effective Damping

This section provides recommendations for the modeling of earthquake-damaged concrete components and systems. This section may be needed if the earthquake-damaged building is being analyzed in its damaged state, or if damaged components are considered in modeling a repair or retrofit. It is not needed for assessment of substantial structural damage or disproportionate earthquake damage (see instead Section 4.4.4).

Modeling for damaged components in DC1 shall follow ASCE/SEI 41, except as modified by the recommendations in Table 5-6. These recommended criteria do not apply to components in DC2. It is assumed here that all prior Performance-Critical Damage (i.e., DC2) has been repaired according to the requirements of Section 5.9. Components classified in DC2 are not considered to exhibit reliable performance in future earthquakes and should not be relied upon without performance-critical repair. Guidance on modeling of components after performance-critical repair is provided in Section 5.9.2.

Table 5-6 Recommendations for Stiffness Modifiers and Damping for Earthquake-Damaged Components and Systems

<table>
<tr><th rowspan="2"></th><th rowspan="2">Smaller Intensity Earthquakes (1)</th><th colspan="2">All Other Earthquakes</th></tr>
<tr><th>Option 1</th><th>Option 2</th></tr>
<tr><td>Stiffness modifier for components experiencing DC1</td><td colspan="2">$\lambda_k = \frac{1}{\mu} \leq 1.0$ (2)</td><td rowspan="2">As per ASCE/SEI 41</td></tr>
<tr><td>Viscous damping ratio</td><td colspan="2">$\zeta = 0.05 + 0.02(M - 1) \leq 0.02$ (3)</td></tr>
</table>

(1) For structural analysis being done for earthquake shaking that produces peak displacement amplitudes less than the peak displacement amplitudes that occurred in the damaging earthquake.

(2) μ is an estimate of the chord rotation ductility from the prior earthquake calculated as a ratio of the peak chord rotation demand in the prior earthquake to the yield chord rotation. Stiffness modifier λ_k is applied to (i.e., reduces) the effective stiffness recommendations in ASCE/SEI 41.

(3) $M = (T_{modified}/T_{initial})^2 > 1$, where $T_{initial}$ is first-mode period calculated for the building using effective stiffness following ASCE/SEI 41 and $T_{modified}$ is first-mode period calculated for the building with component stiffness modified by λ_k as necessary.

Commentary: This section pertains to the analysis of the response of the damaged or repaired building. This section should not be used for the analysis of the building to the damaging earthquake. Such analysis should follow the requirements of Section 3.4.

The modeling recommendations depend on the intensity of the ground motion being used in the analysis. Studies (ATC, 2021a) have shown that stiffness reduction due to prior earthquake damage does not cause appreciable increases in median peak displacement demands in subsequent earthquakes, provided that component damage has not exceeded DS2. An exception arises for performance objectives that involve peak displacement amplitudes that are less than those that occurred in the damaging earthquake (e.g., a serviceability performance objective). In this case, reductions in the effective stiffness of the structural system due to the damaging earthquake may lead to an increase in building vibration period and an increase in peak displacement response. These effects should be incorporated in the structural analysis model by reducing the effective stiffness of the structural system.

Table 5-6 provides recommendations for analysis considering shaking that produces displacement amplitudes less than those experienced during the damaging earthquake. This could be applicable, for example, if the analysis is to estimate how prior damage may have affected response in service-level earthquake shaking. Studies (ATC, 2021a) have shown that prior earthquake shaking damage can result in reduced effective stiffness and increased effective damping that may affect earthquake shaking response at the smaller response amplitudes.

The same models for stiffness modifiers and damping can be used for larger earthquake shaking (Option 1 of Table 5-6), but a simpler approach is also possible. Studies (ATC, 2021a) have shown that prior earthquake shaking damage consistent with DC1 or less does not cause appreciable increase in median peak displacement response in subsequent earthquakes, provided the structural system is one whose behavior is characterized by ductile response without negative post-yield stiffness, and provided the displacement amplitude is greater than the displacement amplitude experienced during the damaging earthquake. Hence, Option 2 in Table 5-6 allows for analysis using the recommendations of ASCE/SEI 41 without modification.

Performance-Critical Damage that has occurred in the damaging earthquake may result in critical strength loss that will affect performance in subsequent earthquakes. Such damage, however, is repaired to restore its strength (as per Section 5.9), such that the strength loss need not be considered when evaluating the repaired or retrofitted building.

As noted in Section 5.9.3, epoxy-injection is not considered a performance-critical repair and should not be used as the sole method of performance-critical repair for any component experiencing DC2. Experimental studies (e.g. Sarrafzadeh, 2021 and Marder et al., 2020) have identified that epoxy-injection can partially restore stiffness of earthquake-damaged components in DC0 and DC1. However, this stiffness restoration is highly dependent on quality of epoxy injection and the axial load on the component. For this reason and due to lack of data, these Guidelines *recommend using the same stiffness for earthquake-damaged components whether epoxy-repaired or not.*

5.8 Criteria to Define Substantial Structural Damage

This section defines gravity-essential components for reinforced concrete systems. As per Condition 2 in Section 4.4.3, if damage classified as DC2 (per Section 5.5) exists in the gravity-essential components listed in Table 5-7 supporting more than 30% of the area of the roof or any individual floor, the building is deemed to have sustained substantial structural damage.

Table 5-7 Gravity-Essential Components in Reinforced Concrete Systems

Gravity-Essential Component
Column classified as nonconforming flexure-shear or shear-controlled per Table 5-5
Wall classified as shear-controlled or nonconforming flexure-controlled per Table 5-5
Beam-column joint with J, BJ, or CJ failure (per Section 5.4.4.3) and at least one exterior face
Slab-column connection without continuity reinforcement (as per ASCE/SEI 41)
Transfer beams

> *Commentary: These conditions are damage in gravity-essential components whose failure compromises the ability of the structure to carry gravity loads. In many cases, these components do not carry sufficient lateral load to trigger the substantial structural damage via Condition 1 of Section 4.4.3, even if a large number of such components are damaged. The triggering damage is DC2, which is associated with the initiation of component strength loss. Although DC2 is a damage classification based on the building's lateral strength loss, the components listed in Table 5-7 may subsequently lose gravity-load-carrying capacity soon after loss of lateral capacity. The 30% limit is in line with the IEBC provisions for substantial structural damage from damage to vertical components carrying gravity loads. Other damage to reinforced concrete systems will be captured by a calculated reduction in lateral-load-carrying capacity in accordance with Section 4.4.4. See also Appendix D.*

5.9 Repair

5.9.1 General

This section describes repair techniques for reinforced concrete structures.

> *Commentary: The focus of this section is on concrete elements reinforced with conventional ductile steel reinforcing bars. The content of the section may not be applicable to prestressed concrete elements or to elements reinforced with other materials.*
>
> *The question often arises of whether it is economic to repair an earthquake-damaged structure. It is beyond the scope of these* Guidelines *to provide a methodology for answering this question. Users are instead referred to the FEMA P-58 suite (FEMA, 2018b) of documents as a potential source of information.*

In accordance with Section 4.4, techniques are described that may be used to restore cosmetic appearance or durability, or to restore strength and deformation capacity. Repairs intended to restore the strength and deformation capacity of components to their pre-earthquake condition are referred to in this document as performance-critical repairs. Some techniques described may also be used as the basis of repair to achieve alternative performance objectives, but this is not a focus of this section.

> *Commentary: The list of repair techniques described in this section is not exhaustive. Exclusion of a technique from this section is not intended to preclude use of that technique unless specifically noted. Regardless of the technique specified, the engineer should carefully consider the technique's ability to achieve the required performance objectives.*
>
> *Detailed examples of repaired concrete components can be found in Repair Test Summaries (Munoz et al., 2024; Sarrafzadeh et al., 2024), which are freely accessible on NHERI DesignSafe and were prepared for users of these* Guidelines*. They provide summaries of over 100 laboratory tests of repaired concrete walls, beams, columns, and joints using a wide range of repair methodologies. The summaries allow for quick identification of tests that can inform the implementation or evaluation of a repair strategy of interest.*

Repairs are not required to restore the stiffness of components to their pre-earthquake condition unless required in project-specific criteria.

> *Commentary: Past studies (ATC, 2021a) have shown that, as long as prior damage does not exceed damage state DS2 (Section 5.5), amplification of drifts in a future design-level ground motion due to prior damage is not anticipated. However, drift amplification may occur in a future service-level earthquake. Refer to Section 5.7 for further discussion.*
>
> *The acceptance by these* Guidelines *of repairs that do not restore the stiffness of components is less restrictive than ACI Code-562,* Assessment, Repair, and Rehabilitation of Existing Concrete Structures *(ACI, 2021), which requires that the stiffness of lateral-force resisting systems should be restored for structures located in Seismic Design Categories B through F as defined by ASCE/SEI 7,* Minimum Design Loads for Buildings and Other Structures *(ASCE, 2022). These* Guidelines *make this concession on the basis that full restoration of stiffness may be impractical, but the engineer may need to consider whether any anticipated stiffness change will affect the functionality of the structure to an unacceptable degree.*

Retrofit of concrete buildings is not covered in this section. Where retrofit of a damaged building is indicated per Chapter 4, reference should be made to the requirements of Section 4.5, other relevant documents (e.g., ASCE/SEI 41, FEMA P-547), and other requirements of the Authority Having Jurisdiction.

5.9.2 Modeling Parameters for Repaired Components

Where there is a need to model a repaired structure, modeling parameters should follow ASCE/SEI 41, except as identified below.

The stiffness of cracked components, or components containing cracks that have been repaired by epoxy injection, should be adjusted as outlined in Section 5.7.

Performance-critical repairs that are identified in these *Guidelines* are intended to restore the pre-earthquake strength and deformation capacity of components. The strength and deformation capacity of components repaired in this manner should be assessed according to the procedures of ASCE/SEI 41.

Repairs that are not performance critical are those intended to restore only cosmetic, durability, or serviceability performance and relates to components in DC0 and DC1. Modeling criteria for components repaired in this manner should be adjusted as outlined in Section 5.7 for damaged components.

> *Commentary:* *As stated, performance-critical repairs are expected to restore the strength and deformation capacity of components. Using common notation, this can be expressed as the repairs achieving* $\lambda_Q = 1.0$ *and* $\lambda_D = 1.0$*. Variation from these values can be expected due to aspects such as quality control in the repair design and construction and specifics of details used. While difficult to quantify, the coefficients of variation for* λ_Q *and* λ_D *are expected to be approximately 15% and 30%, respectively. Larger variation may be expected when replacement of reinforcement is required, although the provisions of Section 5.9.5 have been developed to limit this variability. Refer to Repair Test Summaries (links provided in Section 5.9.1) for specific examples.*
>
> *Consideration may also be required of the impacts of prior strain hardening and strain ageing on repaired components. Strain ageing is only expected to impact mild steels without Vanadium (e.g., grade 40 reinforcement). The significance of this effect depends on the extent of prior reinforcement yielding. Marder et al. (2020) provides some guidance on the subject and notes that epoxy-repaired plastic hinges can exhibit flexural strength increases of up to 25% relative to the strength of identical undamaged components. Prior yielding of reinforcement may also shift the critical section.*
>
> *Consideration should be given to any impact of the increased strength of repaired plastic hinges on the hierarchy of strength and expected collapse mechanism for the structural system. It is not suggested that testing of reinforcement be used to ascertain changes of material properties. Instead, consideration should be given to whether a 25% strength increase in previously yielded components would detrimentally affect performance of the structure.*

5.9.3 Summary of Repair Techniques

Table 5-8 below summarizes whether the repair techniques for concrete and reinforcement that are described in the following sections act as a repair to restore appearance or durability, a performance-critical repair, or both.

Table 5-9 summarizes the types of damage that can be repaired by each technique.

Commentary: General guidance on repair of concrete can be found in ACI 546, Concrete Repair—Guide *(ACI, 2023a), and ACI 562.*

Refer to the Repair Test Summaries (links provided in Section 5.9.1) for examples of combinations of repair techniques used to address specific types of damage to concrete components.

Table 5-8 Summary of Repair Techniques by Ability to Restore Performance

Section	Technique	Repair to restore appearance or durability	Performance-critical repair	Key reference document
5.9.4.1	Concrete replacement	✓	✓	ACI 546
5.9.4.2	Repair using epoxy resin	✓	✗ (1)	ACI 503.7 (2007a); ACI RAP Bulletin 1 (2003); ICRI (2016)
5.9.4.3	Crack sealing	✓	✗	
5.9.5.1	Longitudinal reinforcement replacement	✗	✓	ACI PRC-439.3 (2007b); CRSI (2017)
5.9.5.2	Heat treatment	✗	✓	
5.9.5.3	Longitudinal reinforcement supplementation	✗	✓	
5.9.6	Transverse reinforcement replacement or supplementation	✗	✓	

(1) Repair using epoxy resin may only constitute part of a performance-critical repair where cracking has degraded aggregate interlock.

Table 5-9 Summary of Repair Techniques by Damage Type

Section	Technique	Cracking of concrete	Spalling of concrete	Crushing of core concrete	Damage to reinforcement
5.9.4.1	Concrete replacement	✓	✓	✓	✗
5.9.4.2	Repair using epoxy resin	✓	✗ (1)	✗	✗
5.9.4.3	Crack sealing	✓	✗	✗	✗
5.9.5.1	Longitudinal reinforcement replacement	✗	✗	✗	✓
5.9.5.2	Heat treatment	✗	✗	✗	✓
5.9.5.3	Longitudinal reinforcement supplementation	✗	✗	✗	✓
5.9.6	Transverse reinforcement replacement or supplementation	✗	✗	✗	✓

(1) This does not consider situations where epoxy mortars are used as a patching material.

5.9.4 Repair Techniques for Concrete

Concrete may be damaged by cracking, spalling of cover concrete, or crushing of core concrete, as described previously in this chapter. This section describes repair techniques that address one or more of these types of damage.

5.9.4.1 CONCRETE REPLACEMENT

Concrete replacement can be either a performance-critical repair or a repair to restore appearance or durability. Situations where concrete replacement represents a performance-critical repair include:

- Core concrete has crushed,
- Core concrete has degraded due to abrasion at inclined cracks wider than 2 mm (or about 1/16 in.), and
- Cracking or spalling of cover concrete that may have degraded lap splices or reinforcement anchorage.

Other than the scenario covered by the last bullet above, replacement of cover concrete will generally only be a repair to restore appearance or durability.

> Commentary: *If degradation of lap splices is suspected, consideration should be given to installation of supplementary transverse reinforcement, or mechanical splicing or welding of reinforcement to improve the splice prior to concrete replacement repair.*

Guidance on techniques that may be used to replace concrete can be found in ACI 546.

The specified compressive strength of the replacement concrete should equal or exceed the expected compressive strength of the existing concrete adjacent to the repaired region. Where cover concrete is being replaced, epoxy-patching materials may be suitable. Replacement materials must have structural and thermal properties similar to the existing material. Attention is also required to ensure that concrete replacement does not compromise the durability of the component.

Replacement of core concrete with structural cementitious material constitutes a performance-critical repair provided that reinforcement is not left in a damaged state. Concrete replacement may also be required where repairs to longitudinal reinforcement are required (as per Section 5.9.5), even if the core concrete has not crushed.

> Commentary: *Attention should be paid to ensure that replacement concrete does not adversely change the expected behavior of the repaired component. Use of replacement concrete with higher tensile strength may undesirably result in the component becoming under reinforced, i.e. having a nominal moment strength that is less than the cracking moment. Special mixes, mechanical anchorage, and treatment of adjacent surfaces may be necessary to achieve adequate repair.*
>
> *No prescription is given for the age used as the basis for replacement concrete strength. This age should be selected to suit the requirements of specific projects.*

Consideration should be given to the ability to transfer forces across the boundary between replaced and original concrete. This may commonly include a need to confirm that adequate shear transfer can be achieved by shear friction or other mechanism. Particular attention should be paid to removing all loose concrete from the surfaces of the existing concrete and ensuring the surfaces are clean when replacement concrete is placed.

The most critical aspect of performance when replacing cover concrete is the bond of the repair material to the substrate (Holl and O'Connor, 1997). The bond strength can be evaluated by a pull-off test, as described in ACI 503R (ACI, 2008). The quality of the bond can also be assessed using nondestructive testing techniques such as Impact Echo or Spectral Analysis of Surface Waves (SASW).

5.9.4.2 REPAIR USING EPOXY RESIN

Epoxy resin can be used to repair cracks in concrete components. Depending on the nature of the crack, epoxy resin may be installed by pressure injection or gravity fed pouring.

Epoxy used for injection of damaged concrete need only comply with the requirements for a Type I resin as designed by ASTM C881 (2020).

Epoxies commonly used for repair of cracks can flow into cracks as narrow as 0.05 mm. It is commonly stated that injection of cracks of 0.2 mm or greater width is practical, but narrower cracks may be injected.

> *Commentary: Epoxy manufacturers report their products can flow under pressure into cracks as narrow as 0.05 mm. In practice, 0.2 mm or greater crack widths are commonly injected. Performance requirements for epoxy resin are specified in ASTM C881 (2020). Type I and Type IV resins are intended for bonding hardened concrete to hardened concrete, and consequently, are conceptually relevant to crack injection. Type I is stated to be appropriate for non-load bearing applications while Type IV is stated to be appropriate for load bearing applications.*
>
> *The recommendation that a Type I resin is sufficient for repair of cracks is made on the basis that even Type I resin is required to have tensile and bond strengths that greatly exceed the expected tensile strength of concrete.*
>
> *Pressure injection is effective for repair of cracks on vertical faces, soffits, or top surfaces. Gravity fed pouring can only be used for repair of cracks on top surfaces.*

Extensive guidance exists regarding specifying and implementing epoxy injection repairs (ACI, 2007a; ACI, 2003; ICRI, 2016). Important aspects of implementing epoxy resin repairs include:

- The adequacy of epoxy resin repairs is typically measured against a specified level of resin penetration. Commonly, 80%-90% filling of cracks is deemed to be sufficient.
- The extent of crack filling can be assessed visually using cores taken from repaired cracks.

Where cracks extend through the full thickness of a component, it is generally preferable to seal both sides of the component. However, there are many circumstances in which this will be impractical. In these cases, satisfactory repair may be achieved by use of thixotropic resin.

Epoxy resin alone is insufficient as a performance-critical repair except in the specific circumstance where performance degradation is solely due to degradation of aggregate interlock has occurred due to opening of an inclined crack. In this circumstance epoxy resin can be used as a performance-critical repair provided that the crack width does not exceed 2 mm (about 1/16 in.) and no sliding along the crack is evident.

Guidance on the extent of stiffness restored by epoxy resin can be found in Section 5.7.

5.9.4.3 SEALING OF CRACKS

Sealing of cracks is an appropriate repair technique in circumstances where considerations pertaining to aesthetics or durability are the only imperative for repairing cracks.

Sealing of cracks can be achieved by a number of methods, including:

- Application of elastomeric paint, or
- Use of a flexible sealant, often following grinding or grooving of a chase along the path of the crack to be sealed.

5.9.5 Repair Techniques for Steel Longitudinal Reinforcement

This section describes repair techniques for steel longitudinal reinforcement that has buckled, fractured, or sustained excessive fatigue damage (as determined per Section 5.6).

> *Commentary: It is emphasized that repair of reinforcement is only required for bars that are deemed to be damaged, i.e. to have buckled, otherwise distorted, fractured, or to have sustained excessive fatigue damage. Consideration may be undertaken on a bar-by-bar basis. It is expressly not intended that every bar within regions assigned to DC2 should inevitably be considered damaged.*

For the purpose of this section, longitudinal reinforcement may be taken to also refer to diagonal bars in coupling beams or other reinforcement that is expected to act as a yielding element when a component experiences plastic deformation.

> *Commentary: The repair techniques described here for steel longitudinal reinforcement are also applicable for other forms of damage that, while not caused directly by an earthquake, could be encountered during post-earthquake investigations or repair. Examples include corrosion, or severe damage to bars caused by construction work, such as removal of concrete.*
>
> *Fiber Reinforced Polymer (FRP) should not be used to replace or supplement damaged steel longitudinal reinforcement. This is because of the low strain capacity and brittle behavior of FRP materials.*

5.9.5.1 BAR REPLACEMENT

Bar replacement can be used to repair damage to longitudinal reinforcement.

Replacement bars should have specified strain capacity at least equal to those being replaced. Attention is required to ensure that replacement of bars does not undesirably affect the strength hierarchy and collapse mechanism of the structure. This can be achieved by ensuring that the flexural strength of the element after bar replacement is similar to the original strength. Alternatively, explicit checks should be undertaken to verify that an undesirable mechanism (e.g., formation of a soft story) is not expected to occur during a future earthquake.

The arrangement and detailing of the transverse reinforcement should not be worsened during the repair process. This should be kept in mind when addressing the probable need to modify the dimensions of transverse reinforcement to accommodate the bar discontinuities.

In designing the bar replacement, the engineer must consider:

- Connection technique, and
- Length of bar replacement, including the location of splice or coupler.

Connection Technique

Replacement bars may be connected to the remaining undamaged parts of the existing bars by:

- Welding in accordance with AWS D1.4/D1.4M (AWS, 2018),
- Mechanical connections, or
- Lap splices.

> *Commentary: Determination of which connection type is appropriate will be element specific and may vary between the two ends of a damaged bar section. For example, welding of replacement bars may be required at a wall-foundation interface, with lap splicing being more practical at the upper end of the repair.*
>
> *Many types of mechanical connections are available. These include:*
>
> - Cold-swaged sleeves,
> - Grout-filled sleeves,
> - Steel-filled sleeves,
> - Upset-and-threaded couplers,
> - Tapered-threaded couplers,
> - Sleeve with wedge, and
> - Sleeve with lock screws.
>
> *Mechanical connections are typically large relative to the size of the connected bar. It is important that the engineer consider spacing and cover requirements when specifying such connections. Further guidance on mechanical connections can be found in other publications (ACI, 2007b; CRSI, 2017).*
>
> *Welded connections should preferably be configured to avoid eccentric demands being placed on the existing or replacement bars. This is likely to require the use of either:*

- Butt-welded connections, or
- Indirect butt-welded connections with at least two intermediary pieces equally spaced around the circumference of the bar (as shown in Figure 5-18).

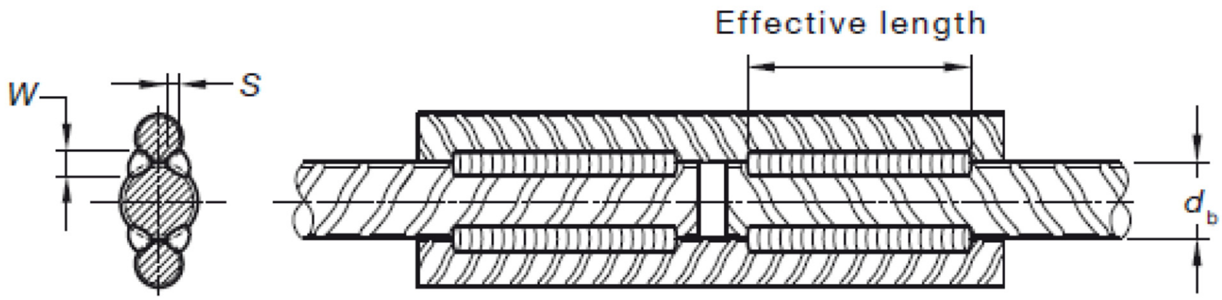


Figure 5-18 Indirect butt-welded connection.

If eccentricity cannot practically be avoided, the potential negative impacts of the eccentricity should be carefully considered. Eccentric welded connections of reinforcement can lead to high concentration of strain demands at the end of the welded splice and a reduction in fracture strain (Motter et al., 2017) and should be avoided where possible. However, successful performance was observed in eccentric welded connections of longitudinal reinforcement in wall components when placed sufficiently outside the plastic hinge zone/damaged zone (Shegay et al., 2023). The concrete surrounding the eccentric connection contributes to balancing the moment created by the eccentricity as shown in Figure 5-19. However, cover concrete cannot reliably provide the balancing force required. For eccentric connections situated adjacent to cover concrete, anchored transverse reinforcement with sufficient strength to balance the eccentric force should be provided.

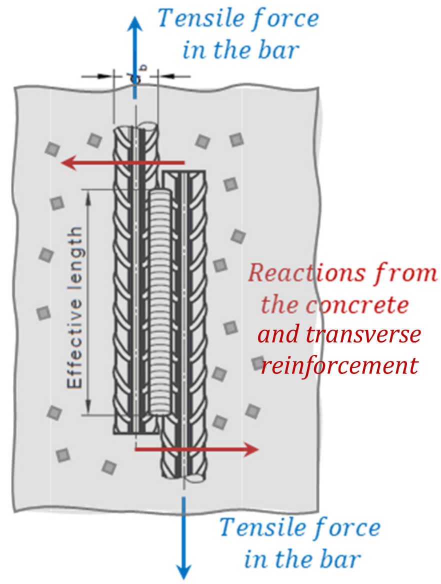


Figure 5-19 Indirect butt-welded connection.

Location and Length of Bar Replacement

Discontinuities created by the bar replacement are not allowed at the critical section.

The positioning of connections between existing and replacement bars should conform with the relevant requirements of ACI 318 regarding splicing of bars in or near plastic hinge regions. Where possible, connections should be staggered along the span of the element.

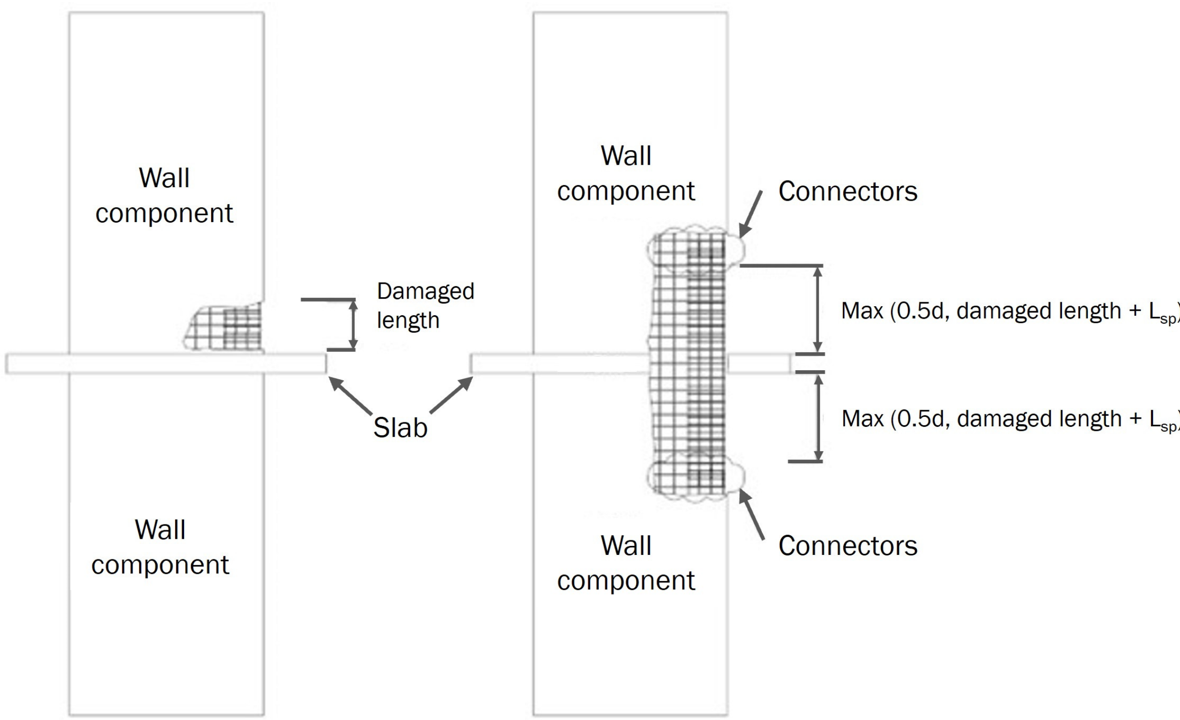
The extent of bar replacement must be sufficient to ensure that the damaged length of bar is replaced. This should be guided by the extent of visual damage and the expected extent of plastic strains in the reinforcement. As shown in Figure 5-20, the length of bar replacement should be at least equal to the greater of:

- Any visibly buckled or distorted length of reinforcement plus an extension to either side of the damaged region equal to the strain penetration length, which may be taken as equal to $0.15f_yd_b$ where the units of f_y are ksi, or $0.022f_yd_b$ if the units of f_y are MPa, and
- A length equal to 0.5 times the section depth to either side of the critical section. However, this length can be reduced if a support (e.g., a foundation or a column supporting a beam) exists at or proximate to the critical section. In such cases, the length of replaced bar need not extend beyond the face of the support by more than the strain penetration length.

> ***Commentary:*** *Connections between reinforcing bars may lead to future plastic deformations being concentrated over a short length. The position of connections should be chosen to avoid this if possible. If such concentration cannot be avoided, the engineer must consider its impact on the future performance of the structure.*

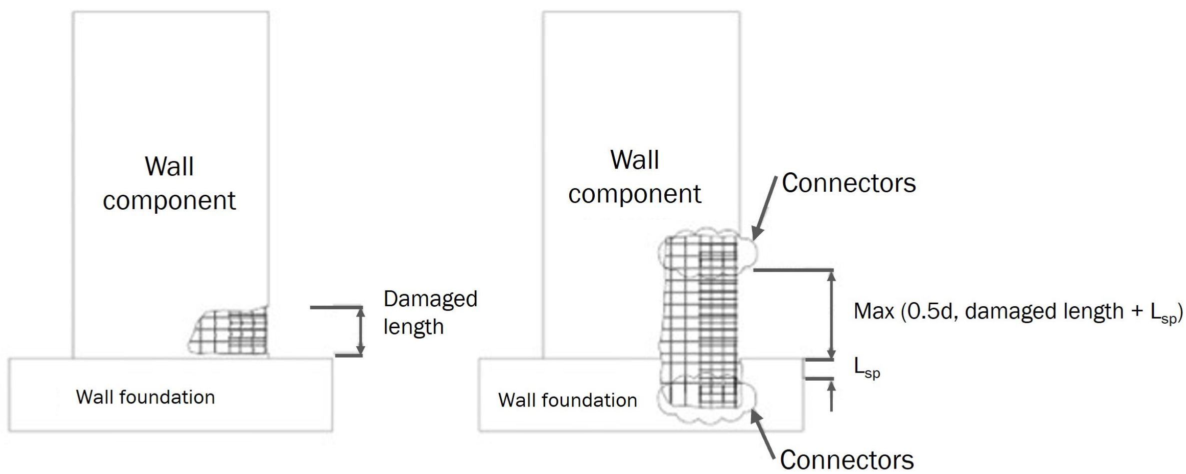
The recommended extent of the bar replacement aims for a repaired critical section with no discontinuities created because of the bar replacement. The connections between the new and original bars are intended to be far enough from the critical section of the repaired component and located in areas where severe damage was not observed during the damaging earthquake. Figure 5-20 shows examples describing the extent of bar replacement in a wall (similar arrangements are recommended for beams and columns). Discontinuities at the critical section created by the bar replacement are not allowed.

Replacement of reinforcement in beam plastic hinges adjacent to columns may be particularly challenging. It may commonly be the case that it is more effective to pass new reinforcement right through the column rather than to try and connect to existing reinforcement within the depth of the column (or heat treatment (Section 5.9.5.2) rather than replacement of beam bars may be more practical).



(a) Damaged region remote from a support (e.g., wall extending past podium diaphragm)

Figure 5-20 Positioning of connections between existing and replacement reinforcing bars (walls shown, but similar arrangements should be used for frame components).



(b) Damaged region adjacent to support

Figure 5-20 Positioning of connections between existing and replacement reinforcing bars (walls shown, but similar arrangements should be used for frame components). (cont.)

5.9.5.2 HEAT TREATMENT

Metallurgical changes resulting from plastic deformation of steel reinforcing bars can be reversed by heat treatment, which can anneal the steel and restore its strain capacity. Specialist metallurgical advice should be sought when considering heat treatment as a method for repairing reinforcing bars

Heat treatment must not be used for bars that have properties created by work hardening. This includes cold-worked steel and quenched and tempered reinforcement. If there is any doubt about the nature of steel requiring repair, the properties of a heat-treated sample should be verified by physical testing.

To be effective, heat treatment must raise the temperature of the steel to its annealing temperature, which is typically approximately 750°C for reinforcing steel. The steel must then be "soaked" at this temperature for a period of time. The required soaking time varies depending on bar size but can be expected to be approximately 1 hour. It must also be ensured that the surrounding concrete is not heated to an extent that is damaging.

> *Commentary: The annealing temperature of reinforcing steel is often taken to coincide with "cherry red heat." The soaking time required and protection of concrete during heating may make this an impractical method of repair in many cases.*

Buckling or other severe straining of reinforcing bars can cause cracks to form in the steel. Where heat treatment is applied to bars that have been straightened after buckling, appropriate investigations should be undertaken to ensure that cracking of the bars has not occurred. Bars identified as containing cracks should be replaced using techniques outlined in Section 5.9.5.1.

5.9.5.3 SUPPLEMENTATION OF EXISTING REINFORCEMENT

Instead of replacing or heat-treating damaged reinforcing bars, it may be possible to install additional bars parallel to the existing reinforcement.

As with bar replacement, supplementary bars should have specified strain capacity at least equal to the existing bars.

It must be ensured that supplementary reinforcing bars are:

- Anchored at each end so that forces can transfer from the bars to other parts of the structure, and
- Enclosed by transverse reinforcement so that they are appropriately restrained and able to engage with shear flow in the component.

> ***Commentary:*** *Supplementation of existing reinforcement could include jacketing of elements.*
>
> *While damaged reinforcing bars are considered unreliable, they are likely to retain significant strength. Consequently, without careful consideration of possible increased flexural strength, repair by supplementation of existing longitudinal reinforcement can result in undesirable structural behavior by causing unintended relocation of plastic hinge regions or changing the strength hierarchy so that shear failure occurs. It may be possible to mitigate this risk by severing the damaged bars, though the remaining severed bars may still lead to concentration of strains over short lengths. If the damaged bars are severed, the recommendations on location and length of bar replacement in Section 5.9.5.1 should be followed.*

5.9.6 Repair Techniques for Steel Transverse Reinforcement

This section describes repair techniques for transverse reinforcement that has been damaged, either during an earthquake or because of repairs to longitudinal reinforcement or core concrete replacement.

Damaged transverse reinforcement can be replaced or supplemented using steel reinforcement, or alternatively using materials such as FRP.

> ***Commentary:*** *Replacement or supplementation of transverse reinforcement using non-ductile materials such as FRP is acceptable because transverse reinforcement is not typically required to sustain large plastic deformations.*
>
> *As with longitudinal reinforcement, damaged transverse reinforcement that is supplemented is likely to retain some capacity. However, in contrast to longitudinal reinforcement, no appreciable risk arises as a result of the potential for this reinforcement to unpredictably increase the shear strength of the element.*

Where FRP or similar materials are used as replacement or supplementary transverse reinforcement, it will generally be preferable to use unidirectional materials with fibers aligned transverse to the component. If materials with fibers are aligned on two or more axes, care should be taken to ensure that strength enhancement on the longitudinal axis does not affect the strength hierarchy of the component and structure.

It is recommended that replacement or supplementary transverse reinforcement be designed to comply with relevant requirements of ACI 318 or ACI 440 (ACI, 2015) when using FRP materials.

Appendix A: Quantification of Inspection Indicators and Performance-Critical Limits

A.1 Overview

This appendix describes the background and development of the methodology for quantifying the Performance-Critical Limits and Inspection Indicators for structural components.

The Performance-Critical Limits (Section 2.2.5.2) are quantitative thresholds that depend on the component type and characteristics, providing a median estimate of the deformation (or force) at DS2. These limits can be compared with deformation (or force) demands from structural analysis to determine if the component has exceeded DS2 during the damaging earthquake. In these *Guidelines,* Performance-Critical Limits are used to support or clarify component Damage Classes (Section 2.2.4.2) in cases where the conclusion based on the observed damage and the Visual Damage State databases is unclear. Such limits are also used to determine Inspection Indicators (Section 3.5.5).

The Inspection Indicators (Section 3.5.5) are quantitative deformation (or force) limits used to identify locations that may potentially have sustained damage. In these *Guidelines,* the Inspection Indicators are used to determine locations requiring detailed inspections when structural analyses are used.

The proposed limits for structural components may be either force based or deformation based, according to the classification given in these *Guidelines*.

A.2 Defining Performance-Critical Limits

This section describes the process for determining the Performance-Critical Limits defined for structural components. For reinforced concrete components, these limits are defined in Section 5.4.

The Performance-Critical Limits identify structural damage that needs performance-critical repair, defining a deformation (or force) limit beyond which the lateral strength and deformation capacity of the component is compromised. The selection of the point of initiation of component strength loss as the critical point is based on extensive study of past earthquake damage, review of experimental data, and analytical studies (Murray et al., 2022; Opabola et al., 2023; Shah, 2021). When components are subject to demands beyond this point, future earthquake performance is substantially impaired, indicating a loss of strength and/or deformation capacity relative to the building’s pre-earthquake condition. As a result, without repair, the component would sustain amplified demands, and a decrease in safety performance relative to its pre-earthquake condition (Murray et al., 2022; Opabola et al., 2024).

For deformation-controlled components, this Performance-Critical Limit is related, but not equivalent to, other measures of component deformation capacity, which may define the point of 20% strength loss or the point of loss of vertical-load-carrying capacity. In Figure A-1, the broken curve defined by points A through E is the generalized load-deformation relationship from ASCE/SEI 41. The deformation at point C is measured by either parameter *a* or *d* depending on the nature of the individual component being represented. Point C is defined in ASCE/SEI 41 as the point at which the component resistance decays by 20% from the peak resistance due to damage in the component. These *Guidelines* define the performance-critical deformation limit for deformation-controlled actions as the deformation at initiation of component strength loss (i.e., DS2), which occurs at some deformation less than the deformation at 20% decay in resistance. The deformation defining the Performance-Critical Limit, i.e., DS2, is given by either $\eta \times a$ or $\eta \times d$.

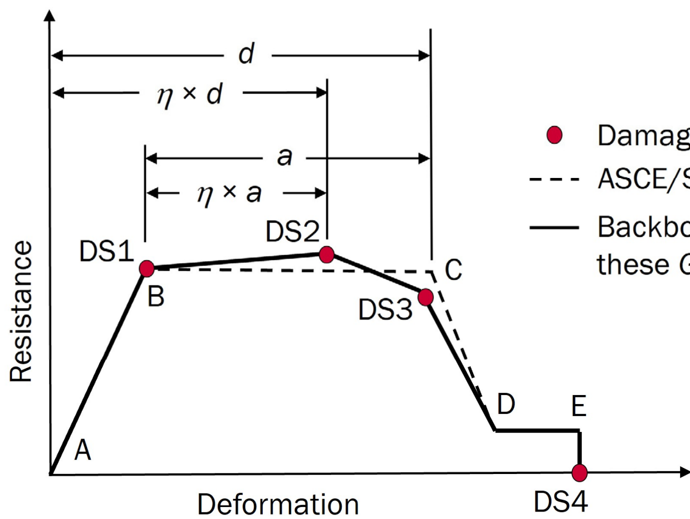


Figure A-1 Generalized force-deformation relationship used in these *Guidelines*, compared to the backbone defined in ASCE/SEI 41. This figure illustrates the definition of η, which defines the Performance-Critical Limit.

The values of η are determined using available experimental data. From experimental data for a component of interest, η is quantified by calculating the ratio of the deformation at the initiation of component strength loss to the deformation at 20% strength loss, as shown in Figure A-2. These calculations were repeated for other experimental tests for the same type of component, providing the basis for the recommended values of η defined in these *Guidelines*.

For reinforced concrete components, these calculations showed that η is typically 0.75 (Section 5.4). For components where 0.75 was not found to be appropriate on the basis of the experimental data, exceptions are provided in Table 5-3.

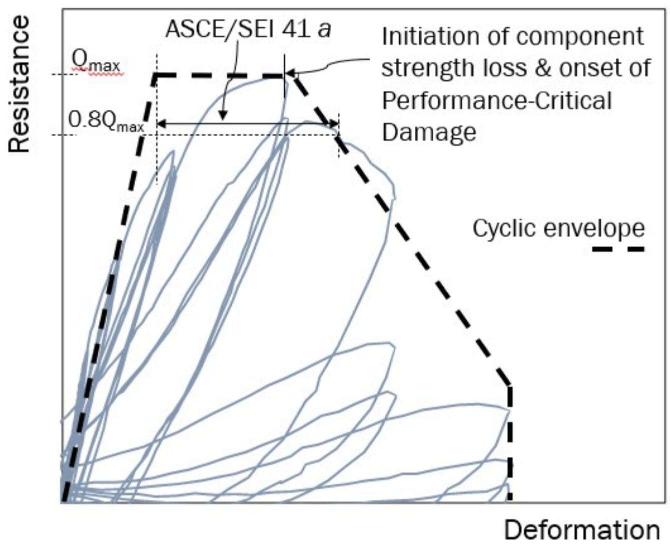


Figure A-2 Illustration of component test force-deformation response data, showing definition of cyclic envelope and point of initiation of component strength loss. This point is also compared to the ASCE/SEI 41 value for *a*. The experimental data are from a reinforced concrete column tested by Sezen (2002), with resistance measured by shear force and deformation by drift ratio.

Given that the onset of component strength loss precedes lateral failure (i.e., 20% lateral strength loss), η should theoretically be ≤ 1.0. However, in some cases, *a* and *d* values in ASCE/SEI 41 are not median estimates and are conservative, producing an implied $\eta > 1$. In these cases, the available test data were used to propose alternative relationships for the median *a* or *d* values, which are defined in Section 5.4.4.2 for coupling beams, columns with conforming transverse reinforcement, and some other components. These *a* and *d* values can be combined with the η values.

Unless alternative procedures are provided in 5.4.4, the performance-critical force-controlled limits are defined as equal to expected strengths, Q_{CE}.

A.3 Defining Inspection Indicators

This section describes the methodology used to derive the quantitative values of inspection indicators, I_p or I_t, of structural components. The inspections indicators are defined in Section 3.5.5 of the *Guidelines.*

The Inspection Indicators are determined such that, if used together with structural analyses to identify a location for detailed inspection, there is a low likelihood of missing damaged components in the inspection. Quantitatively, this definition implies that the probability of Performance-Critical Damage is low, given the demands (force or deformation) on the component corresponding to the Inspection Indicator. The derived values of the Inspection Indicators are based on limiting this probability to approximately 10%.

Based on this definition, the values of the Inspection Indicators for a given component of interest are determined from a reliability analysis accounting for uncertainties both in the demand on and the capacity of the component. The uncertainty in capacity depends on the uncertainty in the Performance-Critical Limit defined in Section A.2. It was determined that this uncertainty, quantified as a logarithmic standard deviation and denoted β_{CSL}, varies from about 0.25 to 0.5, depending on the behavior of the component, as indicated by available test data. Uncertainties in the demand stem from uncertainties in the ground motion, β_{gm}, and structural analyses, β_{model}, from various sources, i.e., the proximity of the structure to a ground motion recording station, availability and reliability of data obtained from building instrumentation, knowledge of material and structural properties, and complexity of the structural analysis used. For example, β_{gm} and β_{model} are lower for instrumented buildings and higher for non-instrumented buildings. β_{gm} is lower for a building near or at a ground motion station. The values of β_{gm} used for derivation of the Inspection Indicators varied from 0.0 for a building with ground motion stations on site to 0.6 for a building without any ground motion stations on a similar site class within approximately 20 km, or about 12 miles (Abrahamson, 2001). The values used for β_{model} varied from 0.1 to 0.2. These values were based on previous reports (e.g., FEMA, 2018b) and the judgment of the project team.

The reliability analysis conducted to determine the Inspection Indicators assumes the random variables representing the deformation parameters (demand and capacity) are statistically independent and lognormally distributed. In structural reliability terms, the limit state function, *G*, is given in Equation A-1 and defines the condition where the median demand on a component, $\tilde{\theta}_D$, exceeds the median capacity defining point of component strength loss, $\tilde{\theta}_{CSL}$:

$$G = \ln\left(\tilde{\theta}_{CSL}\right) - \ln(\tilde{\theta}_D) \tag{A-1}$$

where $\tilde{\theta}_{CSL}$ is the Performance-Critical Limit.

The reliability index, *Z*, accounting for the uncertainties in capacity and demand, can be calculated as:

$$Z = \frac{\ln\left(\dfrac{\tilde{\theta}_{CSL}}{\tilde{\theta}_D}\right)}{\beta_{total}} \tag{A-2}$$

where β_{total} is taken as the square root of the sum of the squares of the underlying sources of uncertainty, including β_{gm}, β_{model}, *and* β_{CSL}.

The reliability index *Z* defines probability, *p*, of $\tilde{\theta}_D$ exceeding $\tilde{\theta}_{CSL}$ .The relationship between *Z* and *p* is defined as:

$$p = \Phi(-Z) \tag{A-3}$$

where $\Phi(.)$ is standard normal distribution function. For $p = 0.10$, consistent with the Inspection Indicator definition, $Z = 1.28$.

In these *Guidelines*, the limit state of interest is the Inspection Indicator, which is defined as I_p or I_t. Replacing $\tilde{\theta}_D$ with I_p or I_t in Equation A-2, I_p (or I_t) can be expressed as:

$$I_p = \frac{\tilde{\theta}_{CSL}}{e^{Z\beta_{total}}} \tag{A-4}$$

where $1/e^{Z\beta_{total}}$ with $Z = 1.28$ is the multiplier with respect to the median value of $\tilde{\theta}_{CSL}$ and defines the fraction of $\tilde{\theta}_{CSL}$ that I_p (or I_t) needs to be equal to, or to exceed, in order to ensure that the probability of missing a damaged component is adequately low. This multiplier is represented by the notation C_i.

$\tilde{\theta}_{CSL}$ represents the Performance-Critical Limit, i.e. $\eta \times a$ or $\eta \times d$. For the values of β_{total} considered, these calculations were used to determine C_i of 0.4, 0.5, or 0.6, as defined in Table A-1 (reproduced from Table 3-1), such that:

$$I_p = C_i\ \eta\ a \tag{A-5}$$

$$I_t = C_i\ \eta\ d \tag{A-6}$$

Table A-1 Inspection Factors Based on Uncertainty in Building Response

Uncertainty	Inspection factor, C_i	Description
Low	0.6	Ground motion instrument available at the building site; analysis model is validated and well developed; component failure modes are well understood
Medium	0.5	More uncertainty in one of the above criteria – for example, ground motion instrumentation on sites within 5 km on the same site class; either analysis model or component failure mode is more uncertain than the criteria for “low uncertainty”
High	0.4	Limited or no nearby ground motion instrumentation; significant uncertainties in structural analysis model or component failure mode

As outlined in Table A-1, establishing the uncertainty is left to the judgment of the engineer, considering: (1) the availability and proximity to measured ground motions to define the seismic demands; (2) whether drawings and other information are available to establish the as-built properties of the structure; and (3) how well the structural analysis model can simulate the expected behavior. The latter point depends on both the characteristics of the structural analysis (e.g., linear versus nonlinear analysis, static versus dynamic analysis, uniaxial plastic hinge versus more detailed fiber or multi-axial models) and confidence in understanding of the structural behavior (e.g., well-controlled yielding of code-conforming components versus nonconforming components where multiple failure modes with rapid onset of degradation are possible).

For more details about how to use C_i in the context of nonlinear vs. linear analysis and force-based limits, refer to Section 3.5.5.

Appendix B: Visual Damage States

B.1 Introduction and Objectives

Visual inspection of earthquake-damaged structural components is an important step in the post-earthquake assessment of buildings. Decisions on the structural condition and the need for structural repair of earthquake-damaged components can be made based on the information collected from the visual inspection process.

To facilitate this inspection process according to these *Guidelines*, VDS databases in the form of spreadsheets were developed to illustrate the progression of damage for various reinforced concrete components, using experimental data from laboratory tests (force-deformation curves) and photographs that document damage as a function of prescribed Damage States and Damage Classes. Table B-1 presents a summary of the VDS databases. In addition to visual damage data, the databases include specimen data such as reinforcement detailing, geometry, material properties, and applied loads to enable engineers to identify test specimens that are representative of the earthquake-damaged components they are assessing. By comparing the damage pattern/level of the earthquake-damaged component in the real building and the damage progression data from the selected representative test specimens in the VDS database, the engineer can efficiently identify the Damage Class of the component and assess the need for structural repair.

Table B-1 Summary of Available VDS Databases

No.	Component	Number of Damage States	Number of Specimens
1	Conforming Flexure-Controlled Walls	5	32
2	Nonconforming Flexure-Controlled Walls	5	19
3	Lap-Splice-Controlled Walls	4	20
4	Shear-Controlled Walls	5	64
5	Shear-Friction-Controlled Walls	4	25
6	Coupling Beams	5	20
7	Slab-Column Connections	5	22
8	Nonconforming Shear-Controlled Columns	4	16
9	Nonconforming Flexure-Shear-Controlled Columns	5	22
10	Conforming Ductile Columns	5	20
11	Lap-Splice-Controlled Columns	4	8
12	Conforming Ductile Beams	5	23
13	Beam-Column Joints (BJ): Interior Joints	5	15
14	Beam-Column Joints (J): Exterior Joints	5	7

B.2 Guidance on Database Usage

B.2.1 Steps to Visually Determine the Component Damage Class

The VDS databases are intended to facilitate identification of the component Damage Class by comparing the damage pattern/level of the earthquake-damaged component and the damage progression data from the databases. The following steps are recommended to assign component Damage Class:

1. **Determine the appropriate database**: Select the relevant database(s) based on the type of component and the failure mode. Table B-2 gives a summary of the criteria used to determine which database to use. If components of interest have characteristics that are roughly within 10% of any criterion or if the damage is inconsistent with the damage characteristics for the selected database, multiple databases should be used to determine the component Damage Class. Expected or measured material properties should be used in strength calculations. For component types not listed in Table B-2, there is only one database, so no criteria are needed. For lap-spliced-controlled walls, the Criterion 1 in Table B-2 assumes that, in the absence of lap-splice failure, the wall is flexure controlled (i.e., $V_{ne}/V_{@Mne} > 1.15$), whereas Criterion 2 is based on: (1) assuming that all wall boundary longitudinal tension reinforcement is lap spliced; therefore, a multiplier of 1.3 is applied for a Class B lap splice (ACI 318-19, Table 25.5.2.1); and (2) applying a multiplier of 1.25 to account for strain hardening (e.g., see ACI 318-19, Section 18.10.2.3(b)). Walls with $l_s/l_d < 1.6$ may or may not be lap-splice-controlled, i.e., damage should be evaluated and compared to representative specimens in other databases to determine the Damage State and Damage Class.

2. **Develop the initial estimate of the component Damage Class**: Review the general description of the DSs on the "Guidance" sheet of the relevant database(s) to develop an initial estimate of the likely DC of the component.

3. **Select representative laboratory test specimens**: Filter the database according to the key parameters and hierarchy described in Section B.2.2. When filtering, select specimens that are within 25% difference of those parameters for the component of interest or, if this is not possible, at least the nearest three specimens. If the component has a non-typical (special/non-conventional) property, such as high concrete compressive strength or high-strength steel, use that parameter as the second filtering criteria. Some parameters may vary during the earthquake, e.g., axial load, and, therefore, a range of values should be considered.

4. **Determine the component Damage Class**: Compare the damage pattern/level of the earthquake-damaged component and the damage progression data of the selected representative specimens. Databases relate DS and DC on the top headers on each database. Thus, once it is determined that a DS photograph is similar to the damage of the component of interest, check the top headers to determine the DC.

For beams, a VDS is provided only for conforming (ductile) beams. For other beams, Section 5.5 provides general damage descriptions that can be used to identify the DC. In some situations, the column VDS may be used, filtering for columns only with low axial load. However, this approach may not be appropriate in all cases, due to the asymmetry of beam longitudinal reinforcement (unlike columns) and detailing and development-length issues specific to beams.

Table B-2 Criteria for Determining the Appropriate VDS Database

Component Type	Criterion 1	Criterion 2	Database
Walls	$V_{ne}/V_{@Mne} > 1.15$ and $V_{nfe} > V_{@Mne}$	$s/d_b \leq 8.0$, and $A_{sh,provided}/A_{sh,required} \geq 0.7$	1. Conforming Flexure-Controlled Walls
		$s/d_b > 8.0$, and/or $A_{sh,provided}/A_{sh,required} < 0.7$	2. Nonconforming Flexure-Controlled Walls
		$l_s/l_d < 1.625$	3. Lap Splice-Controlled Walls
	$V_{ne}/V_{@Mne} \leq 1.15$ and/or $V_{nfe} \leq V_{@Mne}$	$V_{ne} < V_{nfe}$	4. Shear-Controlled Walls
		$V_{ne} \geq V_{nfe}$	5. Shear-Friction-Controlled Walls
Columns	$V_p/V_o > 1.0$	N/A	8. Nonconforming Shear-Controlled Columns
	$0.6 < V_p/V_o \leq 1.0$	N/A	9. Nonconforming Flexure-Shear-Controlled Columns
	$V_p/V_o \leq 0.6$	ACI 318 conforming seismic details	10. Conforming Ductile Columns
	$l_{splice}/l_d < 1.625$	N/A	11. Lap Splice-Controlled Columns

where:
V_{ne} = Expected shear capacity per ACI 318, using expected material properties
M_{ne} = Expected flexural capacity per ACI 318, using expected material properties
$V_{@Mne}$ = Shear demand at expected flexural capacity, using expected material properties
V_{nfe} = Expected shear friction capacity per ACI 318, using expected material properties
l_s = Length of splice provided
l_d = Development length per ACI 318, using expected material properties
V_p = Shear demand at expected flexural strength (for columns)
V_o = Expected undegraded shear capacity, according to ASCE/SEI 41 and ACI 369

B.2.2 Key Parameters to Identify Representative Test Specimens

Table B-3, Table B-4, Table B-5, and Table B-6 present key parameters to identify representative test specimens, including a hierarchy approach for filtering. The filtering hierarchy (number in the parentheses) suggests the most important parameters to use in the filtering process, recognizing that the number of filtering parameters needed depends on the characteristics of the earthquake-damaged component.

Table B-3 Key Parameters to Identify Representative Wall Tests

Database	$l_w c/b^2$	s/d_b	$A_{sh,provided}$ / $A_{sh,req}$	$V_{ne}/V_{@Mne}$	ρ_{wh}	$V_{ne}/V_{@nfe}$
1. Conforming Flexure-Controlled Walls	✓ (1)	✓ (3)		✓ (2)		
2. Nonconforming Flexure-Controlled Walls	✓ (1)	✓ (2)	✓ (3)			
4. Shear-Controlled Walls	✓ (3)			✓ (1)	✓ (2)	
5. Shear-Friction-Controlled Walls	✓ (3)			✓ (2)		✓ (1)

Table B-4 Key Parameters to Identify Representative Frame Component Tests

Database	V_p/V_o	ρ_t	$P/A_g f'_{ce}$	ρ_l	a/d
8. Nonconforming Shear-Controlled Columns	✓ (1)	✓ (2)	✓ (3)		
9. Nonconforming Flexure-Shear-Controlled Columns	✓ (1)	✓ (2)	✓ (3)		
10. Conforming Ductile Columns		✓ (3)	✓ (2)	✓ (1)	
12. Conforming Ductile Beams		✓ (3)		✓ (1)	✓ (2)

Table B-5 Key Parameters to Identify Representative Spliced-Controlled Component Tests

Database	l_s/d_b	$A_{vsp} f_{yet}$ / $A_{sl} f_{yel}$	c_b/d_b
3. Lap-Splice-Controlled Walls	✓ (1)	✓ (2)	✓ (3)
11. Lap-Splice-Controlled Columns	✓ (1)	✓ (2)	✓ (3)

Table B-6 Key Parameters to Identify Representative Other Component Tests

Database	l_n/h	$V_g/\phi V_o$	$P/A_g f'_c$	h_b/h_c	V_u / V_{nj}	s_{joint}/h_c
6. Coupling Beams	✓ (1)					
7. Slab-Column Connections		✓ (1)				
13. Beam-Column Joints (BJ) *Interior Joints			✓ (1)		✓ (3)	✓ (2)
14. Beam-Column Joints (J) *Exterior Joints			✓ (1)	✓ (2)		

B.2.3 Additional Guidance to Determine the Component Damage Class

This section provides additional information for interpreting the information in the VDS databases.

- Descriptions under each photograph indicate the drift at which each photograph was taken. Other information may be provided in the descriptions to help interpret what is observed in the photographs, such as that there was spalling at the boundaries. If the photograph is at a residual displacement, it should be noted that some cracks may have closed.

- Photographs reported in databases are the photographs that were nearest to specific DS; they are not necessarily at the exact DS. To aid in interpretation, the databases (except joint databases) provide DS1 and DS2 ratios, which are ratios that relate the deformation at which the photograph was taken to the deformation at DS1 and DS2, respectively. It is likely that a progression of photographs will need to be reviewed to identify the DC.

- Earthquake-damaged components with parameters that are near the boundaries of the parameters (criteria) used to establish the databases (e.g., conforming and nonconforming walls): In this situation, it may be necessary to review information in more than one database. For example, a wall classified as a shear-friction-controlled wall (according to calculations) might have damage consistent with a diagonal-tension shear failure, in which case, the shear-controlled wall database should be used. Thus, for components that are within 10% of any criterion used to distinguish between two databases, both databases should be used to determine the component DC.

- After filtering to identify representative test specimens, check all parameter values of the selected specimens and, if possible, avoid using specimens with properties considerably different than the parameters of the earthquake-damaged component (e.g., if the filtering parameter is “high strength concrete” and the selected specimen used “low strength concrete”).

B.2.4 Residual Diagonal Crack Width Indicative of DC2

Residual crack widths are not reliable indicators for assessing the residual capacity of flexure-controlled concrete members. However, residual crack widths might provide useful information on the DC of shear-controlled components. A database of shear-controlled components with measured residual crack width data following a deformation demand that triggered the onset of component strength loss (i.e., DC2) was collated. The database consists of eight unreinforced beam-column joint specimens, four flexure-shear-controlled columns, four shear-controlled columns, and eight shear-controlled walls.

Given the small size of the databases, it was not possible to quantify the influence of a wide range of parameters on the residual crack width indicative of DC2 for each component type. A simple probabilistic approach was adopted in defining residual crack width thresholds indicative of DC2. First, a cumulative distribution function is fitted to the residual crack width at initiation of component strength loss, i.e. DS2, for each component type. The cumulative distribution function (CDF) for each component type is presented in Figure B-1. Due to the size of flexure-shear controlled column dataset, both flexure-shear and shear-controlled column datasets were merged to develop the cumulative distribution function in Figure B-1b. It is also noted that the maximum stirrup spacing-to-effective-depth ratio (s/d) in the column dataset is 0.6. Hence, Figure B-1b may not apply to lightly confined columns (i.e., $s/d \geq 0.75$). Based on an assumption that the critical diagonal failure plane in columns with $s/d \geq 0.75$ may cross zero or one stirrup, it is recommended that the cumulative distribution function for unconfined joints should be adopted for columns with $s/d \geq 0.75$. Additional test data are needed to refine Figure B-1.

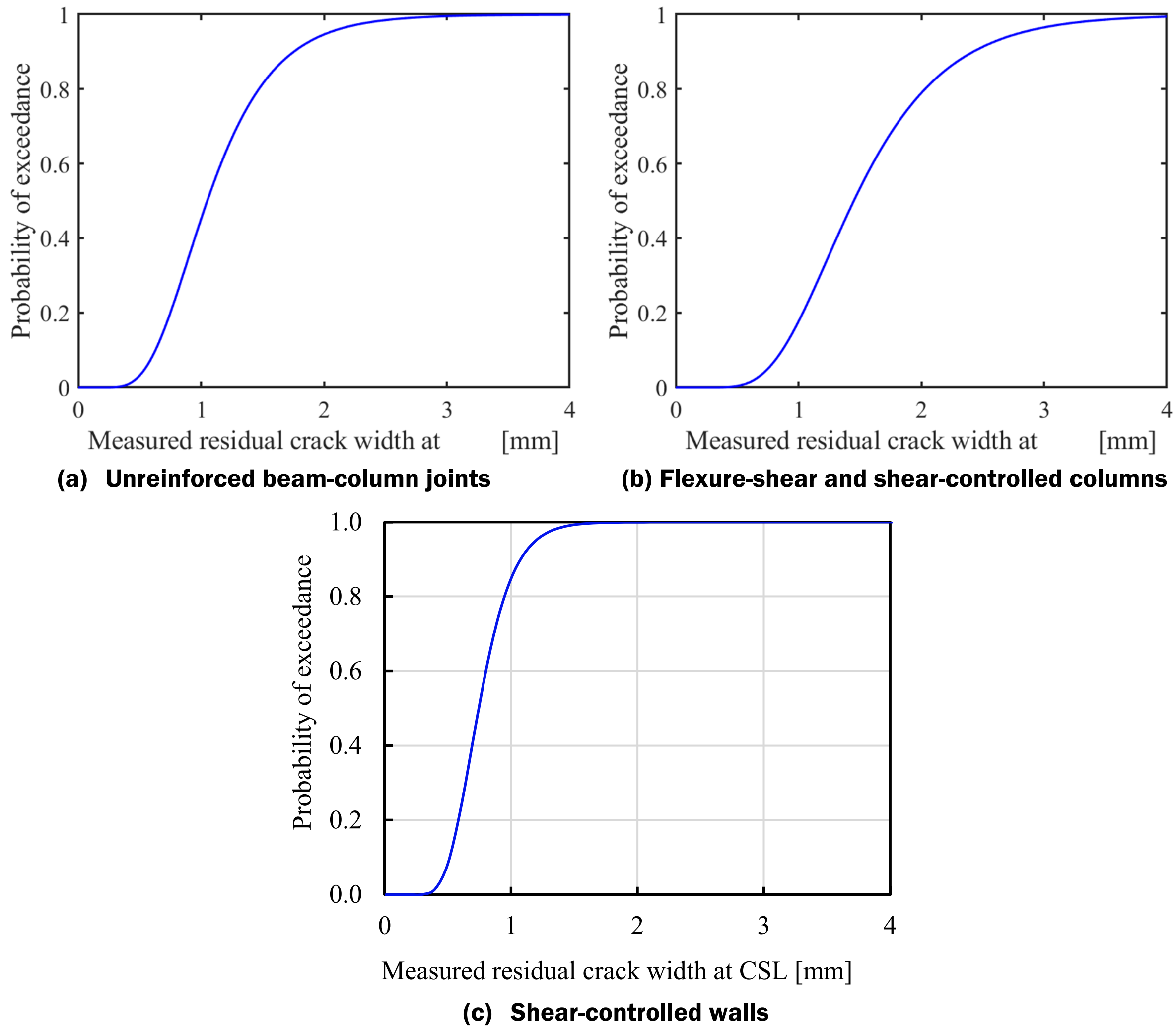


Figure B-1 Cumulative distribution function for measured residual crack width at the onset of component strength loss (CSL). Only diagonal residual crack width data were used in developing the cumulative distribution functions.

Once the CDFs were developed, a residual crack width limit was defined from the CDF based on a high probability that the residual crack width corresponds to DC2. This appendix adopts 80% probability of exceedance or higher for defining the residual crack width limit. The selected residual crack width limits are presented in Table B-7 for each component type. For shear-controlled walls, based on Figure B-1, a 1 mm crack width has 85% probability of reaching DS2; however, the recommended residual diagonal crack width limit was increased to 1.5 mm based on judgment due to the limited data available.

Table B-7 Recommended Residual Diagonal Crack Width Limits Indicative of DC2

Component Type	Residual diagonal crack width limit (mm) [1]
Unreinforced beam-column joint	≥ 1.5 (about 1/16 in.)
Shear-controlled columns ($s/d \geq 0.75$)	≥ 1.5 (about 1/16 in.)
Flexure-shear and shear-controlled columns ($s/d < 0.75$)	≥ 2.0 (about 1/16 in.)
Shear-controlled walls	≥ 1.5 (about 1/16 in.)

(1) If observed, these residual crack widths indicate a high probability of DC2. Maximum residual crack widths narrower than the recommended limits do not preclude DC2.

It is important to note that a maximum residual crack width narrower than the recommended limits in Table B-7 does not indicate that a component can be classified as DC1 or DC0. If the measured residual crack width is narrower than the recommended limits in Table B-7, other relevant checks must be conducted before concluding that the component is classified as DC1 or DC0.

B.3 Description of Key Damage States

B.3.1 General Description

All VDS databases have at least four Damage States that are related to the Damage Classes (Figure B-2 and Table B-8), where Damage Classes are consistent with Chapter 4, as described below (* indicates that these values may not exist in all VDS):

- *DS0 – Damage State corresponding to the attainment of $0.7V_{max}$ on the force-displacement backbone. DS0 represents a damage state in the elastic range of the component response.
- DS1 – Damage State corresponding to the effective yield (initiation of nonlinear behavior) of the component. From the experimental studies, the effective yield is defined by drawing a secant line from the origin to pass through the backbone curve at 70% of maximum lateral load (V_{max}). The point of intersection between that secant line and the horizontal line corresponding to V_{max} is the effective yield point.
- *DS1.b – Damage State corresponding to inelastic response between DS1 and DS2. This Damage State is provided in databases where there is a significant difference in the deformation at DS1 and DS2. In these cases, an additional photograph is provided to help define the DC of the component.
- DS2 – Damage State corresponding to the deformation at initiation of component lateral strength loss. A deformation equal or higher than this Damage State corresponds to DC2, i.e. Performance-Critical Damage.

- DS3 – Damage State corresponding to the deformation at onset of lateral failure, which is defined in experimental studies as deformation corresponding to 20% drop in peak lateral strength.

- DS4 – Damage State corresponding to drift when the experimental study reported loss of initial axial capacity and/or at which the lateral resistance has degraded to zero.

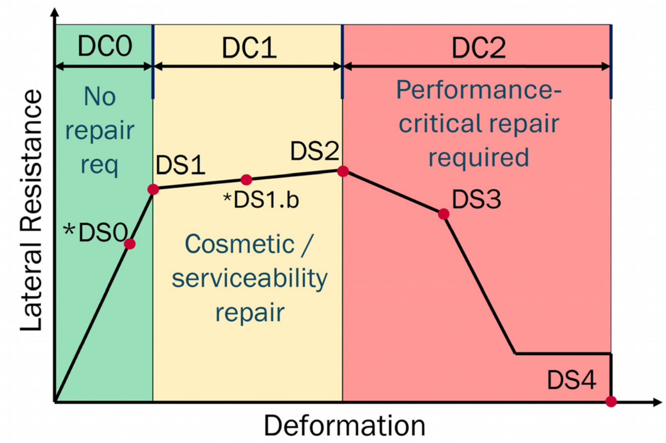


Figure B-2 Relation between Damage States and Damage Classes on a component backbone. An asterisk indicates that these values may not exist for all VDS.

Table B-8 Damage States Used in the VDS Databases

No.	Component	DS0	DS1	DS1.b	DS2	DS3	DS4
1	Conforming Flexure-Controlled Walls	✗	✓	✓	✓	✓	✓
2	Nonconforming Flexure-Controlled Walls	✗	✓	✓	✓	✓	✓
3.	Lap-Spliced-Controlled Walls	✓	✗	✗	✓	✓	✓
4	Shear-Controlled Walls	✓	✓	✗	✓	✓	✓
5	Shear-Friction-Controlled Walls	✗	✓	✗	✓	✓	✓
6	Coupling Beams	✗	✓	✗	✓	✓	✓
7	Slab-Column Connections	✗	✓	✓	✓	✓	✓
8	Nonconforming Shear-Controlled Columns	✓	✗	✗	✓	✓	✓
9	Nonconforming Flexure-Shear-Controlled Columns	✓	✓	✗	✓	✓	✓
10	Conforming Ductile Columns	✗	✓	✓	✓	✓	✓
11	Lap-Spliced-Controlled Columns	✓	✗	✗	✓	✓	✓
12	Conforming Ductile Beams	✗	✓	✓	✓	✓	✓
13	Beam-column Joints (BJ): Interior Joints	✗	✓	✓	✓	✓	✓
14	Beam-column Joints (J): Exterior Joints	✗	✓	✓	✓	✓	✓

B.3.2 An Example of Damage States and Information Provided

To illustrate the information provided in the VDS databases, this section presents an example of the DSs used in the conforming flexure-controlled wall database. For conforming flexure-controlled walls, the DSs can typically be classified as follows, and are illustrated in Figure B-3.

- DS1 – Characterized by formation of horizontal cracks initiating from the extreme edges of the wall with or without diagonal (tension) shear cracks in the web, followed by sequential yielding of longitudinal reinforcement until the effective yield strength of the wall is reached.
- DS1.b – Characterized by spalling of concrete cover at the extreme fibers (boundary elements) of the wall.
- DS2 – Point of initiation of component strength loss; commonly characterized by buckling of the boundary longitudinal reinforcement closest to the wall edge.

- **DS3** – A 20% component strength loss from peak strength; characterized by fracture of buckled longitudinal bars near the wall edge combined with concrete core crushing, opening or fracture of hoops and crossties, or local instability at the wall boundary. For walls with shear stress demands exceeding ~ $6\sqrt{f_c^{'}(psi)}$), 20% loss of component strength may result from web crushing (crushing along a diagonal strut). In barbell-shaped wall cross sections, web crushing may occur in the web adjacent to the boundary columns.

- **DS4** – Loss of axial load-carrying capacity or total loss of component strength as a result of sequential fracture of longitudinal bars and concrete crushing for walls with low compression demands and squat cross-sections (i.e., $l_w c/b^2 < 20$ or 15), or crushing of concrete or out-of-plane instability across the entire wall length for walls with significant compression demands and slender cross-sections ($l_w c/b^2 < 60$ or 70), leading to abrupt loss of axial-load-carrying capacity.

This information is provided on the Guidance sheet of the relevant database. Note that in describing the experimental tests, the expected material properties f'_{ce} and f_{ye} are those measured during the testing program.

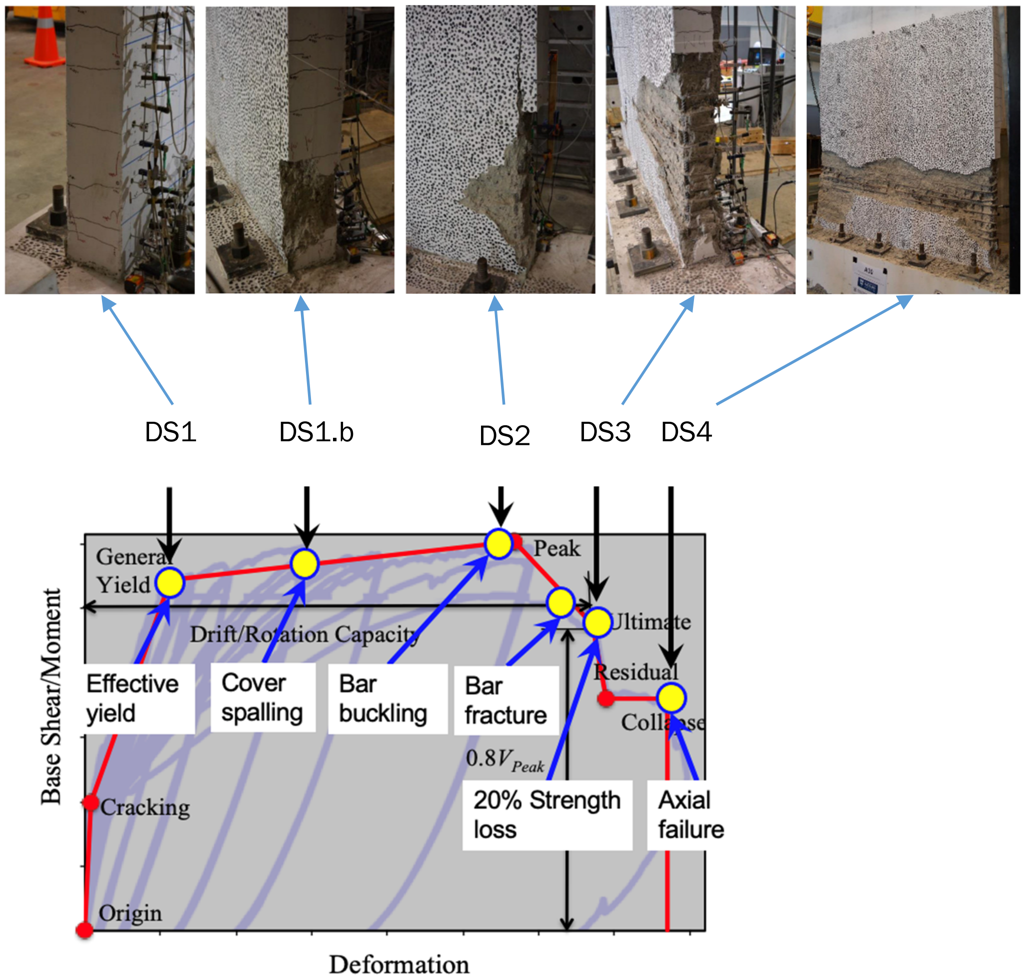


Figure B-3 **Backbone relationship showing Damage States for a conforming flexure-controlled wall.**

B.4 VDS Databases

The VDS databases are provided in spreadsheet files. The main criterion for selecting the experimental tests for inclusion in each dataset was the availability of good quality photographs at the various prescribed Damage States. Most of the data are from test specimens subjected to quasi-static, reversed cyclic loading; although there are a limited number of dynamic or non-reversed cyclic tests included. The data included in each set are described in this section.

B.4.1 Conforming Flexure-Controlled Walls

B.4.1.1 DESCRIPTION OF DATABASE

A dataset of 62 code-conforming flexure-controlled wall tests was assembled using information from the comprehensive database of reinforced concrete structural wall tests reported in the literature (Abdullah, 2019; Abdullah and Wallace, 2019).

Out of the 62 walls in the dataset, photographs at the DSs described in Section B.3.2 for 32 wall tests were collected and reported in the dataset. If available, two photographs for each DS were included, showing both an overall view of the wall and a close-up view (usually of the boundary). The characteristics of those 32 walls are summarized through histograms for several dataset parameters for the 32 tests in Figure B-4, where $P/A_gf'_{ce}$ is the compressive axial load normalized by the measured concrete compressive strength (f'_{ce}) and gross concrete area (A_g), M/Vl_w is the ratio of base moment-to-base shear normalized by wall length (l_w).

Figure B-5(h) shows the slenderness parameter l_wc/b^2 distribution of the conforming dataset, where l_w is the length of the wall, c is the depth of neutral axis corresponding to a concrete compressive strain of 0.003, and b is the width of the flexural compression zone.

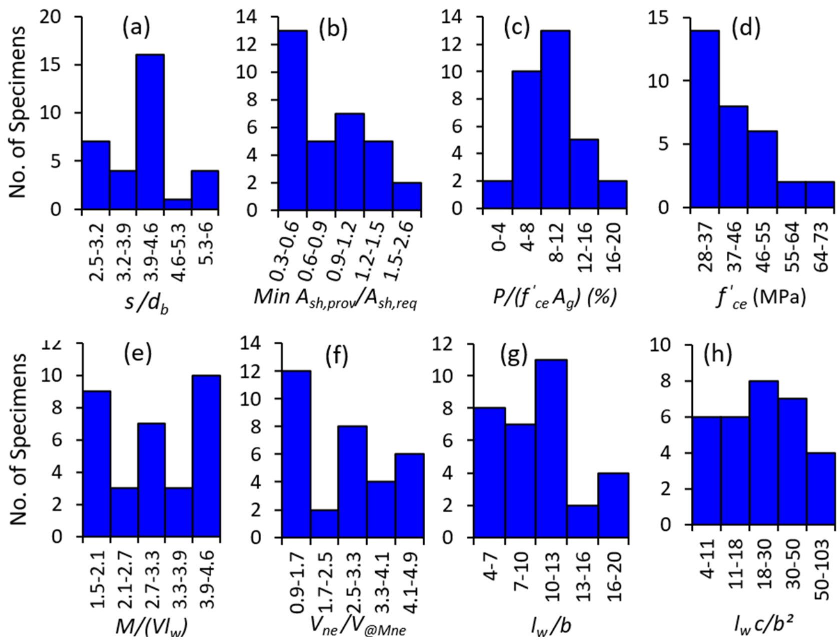


Figure B-4 Histograms of the conforming flexure-controlled walls VDS database.

B.4.1.2 GUIDANCE ON IDENTIFYING REPRESENTATIVE TESTS

Figure B-5a indicates that there is a significant correlation between rotation capacity and a slenderness parameter, $(l_w/b)(c/b) = l_wc/b^2$. This parameter provides an efficient means to account for the slenderness of the cross section (l_w/b) and the slenderness of the flexural compression zone of the cross section (c/b) on the deformation capacity of conforming flexure-controlled walls (Abdullah and Wallace, 2019 and 2020).

Walls with values of l_wc/b^2 lower than 10 tend to be flexure-tension controlled and generally have large deformation capacities, whereas walls with values of l_wc/b^2 exceeding 70 (slender cross-section and deep compression zone) tend to be flexure-compression controlled and generally have low deformation capacities and simultaneous occurrence of lateral and axial failures (Abdullah and Wallace, 2021). Therefore, the value l_wc/b^2 of the wall should be estimated and used to identify representative wall tests (e.g., calculated $l_w c/b^2 \pm 5$). If further refinement is desired to identify the most representative tests, the longitudinal bar slenderness ratio, s/d_b, should be used because the initiation of component strength loss typically coincides with initiation of longitudinal bar buckling, which is moderately impacted by s/d_b (Figure B-5b).

The variables in $l_w c/b^2$ are related to geometry (i.e., l_w and b) and are readily available, except for c. To filter specimens for use in the VDS databases, Equation B-1 can be used to compute the approximate depth of neutral axis, c:

$$\frac{c}{l_w} = k_1 + k_2 \frac{P}{A_g f'_{ce}} \tag{B-1}$$

where values of k_1 and k_2 are obtained from Table B-9 based on the cross-section shape of the wall. Equation B-1 is derived based on data from 696 walls with $P/(A_g f_{ce}') > 0$ (Abdullah and Wallace, 2020). The first term considers the impact of longitudinal reinforcement (ratio and strength) and concrete strength, whereas the second term addresses the impact of axial load. Figure B-5c compares the depth of neutral axis computed from Equation B-1 with that computed from detailed sectional analysis.

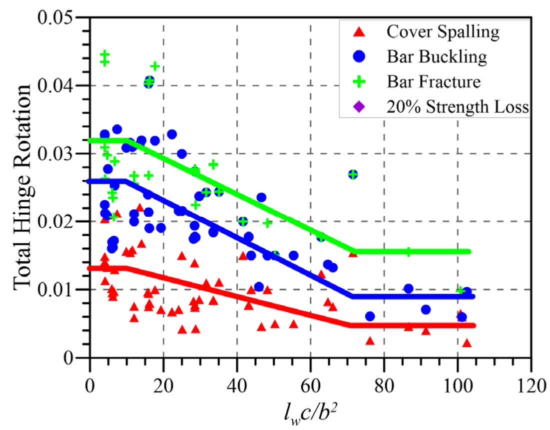


Figure B-5a Comparison of rotation capacity of the dataset at key damage states.

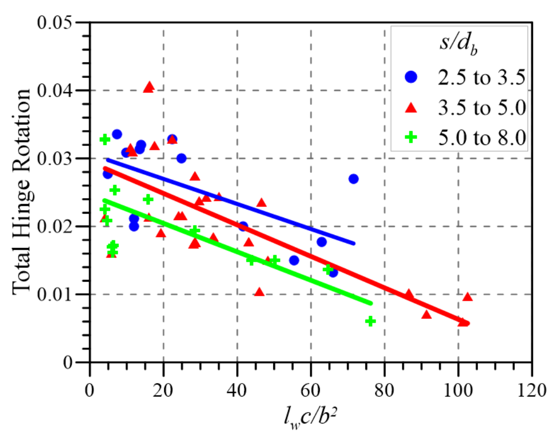


Figure B-5b Influence of s/d_b on total hinge rotation of conforming flexure-controlled walls.

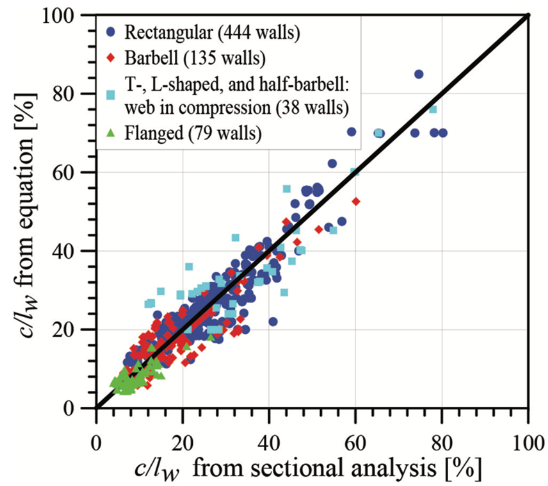


Figure B-5c **Comparison of *c* computed from Equation B-1 with that from detailed sectional analysis.**

Table B-9 **Neutral Axis Depth Parameters Used in Equation B-1**

Wall cross-section shape	k_1	k_2
Rectangular	0.10[1]	1.2
Barbell and flanged	0.03	1.4
T-, L-shaped, and half-barbell (flange in compression)	0.03	0.7
T-, L-shaped, and half-barbell (web in compression)	0.20	2.0

(1) This value is for walls with longitudinal reinforcement concentrated at the wall boundary. For walls with uniformly distributed reinforcement, k_1 = 0.05 and 0.20 when longitudinal reinforcement ratio is < 0.005 and ≥ 0.015, respectively. For intermediate values, linear interpolation can be applied.

B.4.2 Nonconforming Flexure-Controlled Walls

B.4.2.1 DESCRIPTION OF DATABASE

A database of 19 nonconforming flexure-controlled wall tests was created from 208 wall tests. Histograms for several dataset parameters for the 19 tests are shown in Figure B-6. Most of the tests in the database do not have special boundary elements (i.e., ACI 318-19 is not satisfied) and are walls with s/d_b ratios higher than 8 (Figure B-6 (b) and (a), respectively).

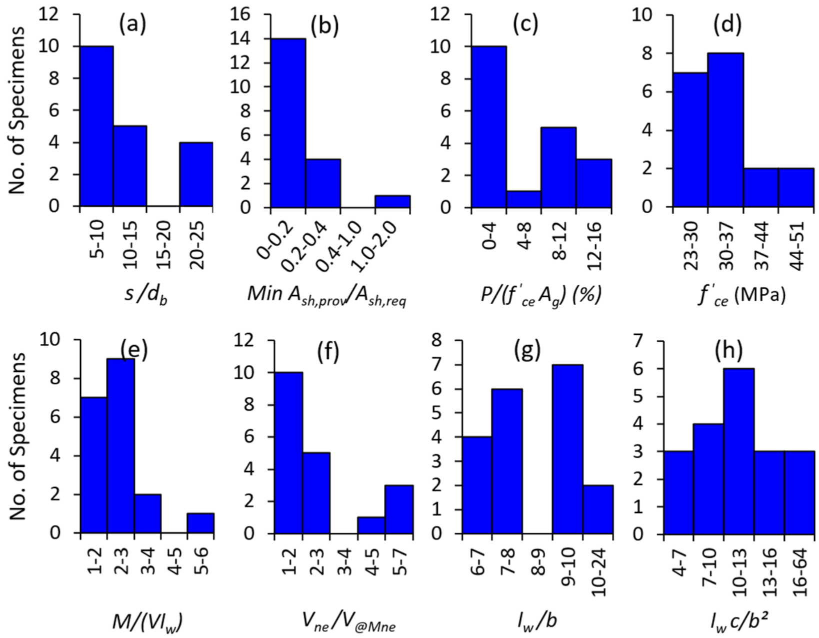


Figure B-6 Histograms of the nonconforming flexure-controlled walls VDS database.

B.4.2.2 GUIDANCE ON IDENTIFYING REPRESENTATIVE TESTS

The key parameters to identify representative tests are the slenderness parameter ($l_w c/b^2$) and the longitudinal bar slenderness ratio (s/d_b), which are the same key parameters described in detail in Section B.4.1.2. In addition, the boundary transverse reinforcement confinement ratio may be a key parameter if boundary transverse reinforcement is widely spaced and/or poorly configured. The boundary transverse reinforcement confinement ratio is defined as $A_{sh,provided}/A_{sh,required}$, where $A_{sh,provided}$ is the transverse reinforcement provided in boundary elements and $A_{sh, required}$ is the transverse reinforcement required by ACI 318.

B.4.3 Lap-Splice-Controlled Walls

B.4.3.1 DESCRIPTION OF DATABASE

A database of 20 lap-splice-controlled wall tests was created from 39 wall tests. Histograms for several dataset parameters for the 20 tests are shown in Figure B-7. Most of the tests in the database have a ratio of the splice-length-provided to diameter-of-longitudinal-reinforcement equal to or less than 40.

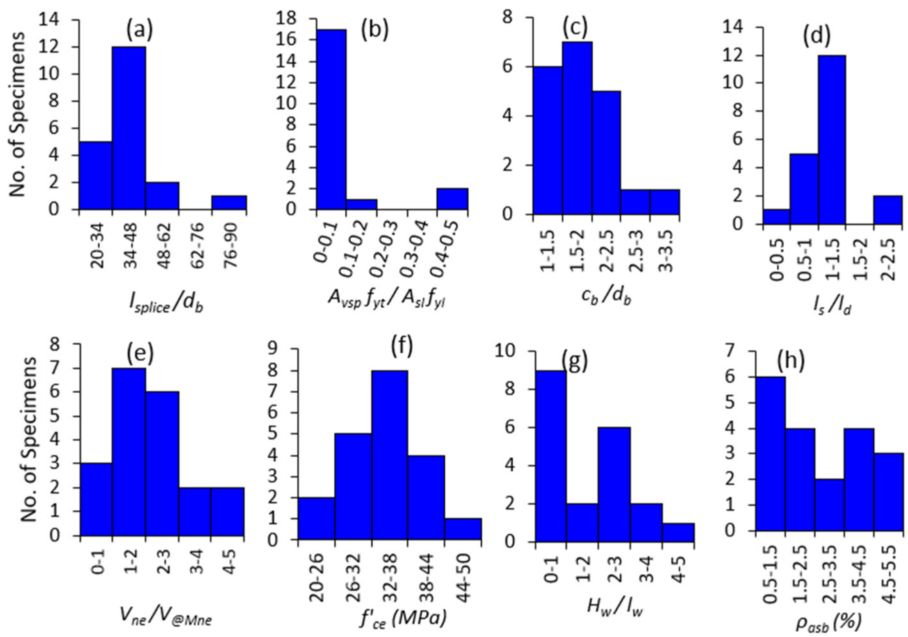


Figure B-7 Histograms of the lap-splice-controlled walls VDS database.

B.4.3.2 GUIDANCE ON IDENTIFYING REPRESENTATIVE TESTS

The key parameters to identify representative tests are the ratio of the lap splice provided to the longitudinal reinforcement diameter (l_s/d_b), the ratio of confining force to the yield strength of spliced bars ($A_{vsp}f_{ye}/A_{sl}f_{ye}$), and the cover to longitudinal reinforcement diameter (c_b/d_b).

B.4.4 Shear-Controlled Walls

B.4.4.1 DESCRIPTION OF DATABASE

A database of 64 tests exhibiting flexure-shear, diagonal-tension, or diagonal-compression wall behavior was assembled in the shear-controlled wall database. Histograms for several dataset parameters for the flexure-shear wall tests are shown in Figure B-8 and for the diagonal-compression/tension-controlled tests in Figure B-9. Most of the specimens are walls with a nominal shear-to-flexure strength ratio ($V_{ne}/V_{@Mne}$) between 0.7 and 1.2 and a horizontal web reinforcement ratio less than 1%, as shown in Figure B-8 (f) and (d) and Figure B-9 (f) and (d), respectively.

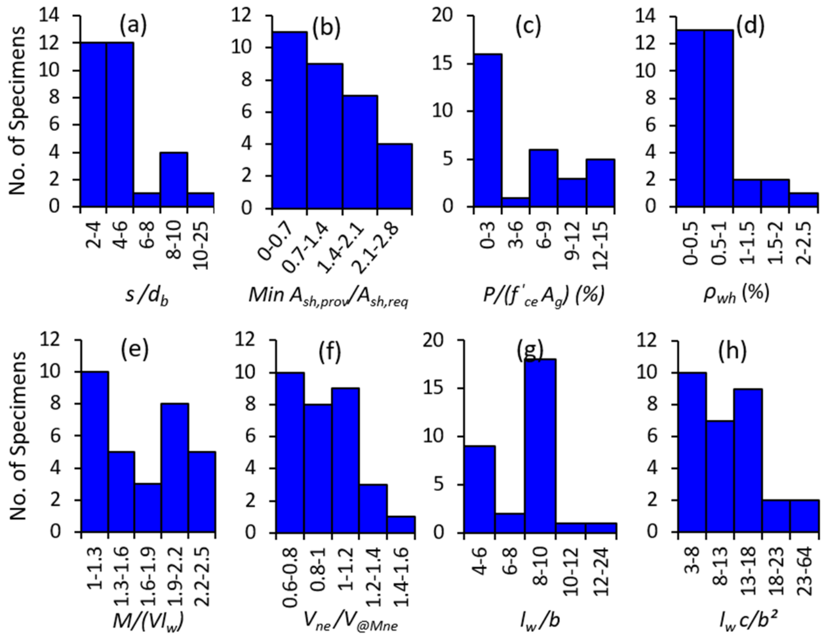


Figure B-8 Histograms of the flexure-shear walls VDS database.

B.4.4.2 GUIDANCE ON IDENTIFYING REPRESENTATIVE TESTS

The key parameters to identify representative tests are the slenderness parameter ($l_w c/b^2$) and the longitudinal bar slenderness ratio (s/d_b), which are described in detail in Section B.4.1.2. In addition, the nominal shear-to-flexure strength ratio and the horizontal-web-reinforcement ratio may be key parameters. The nominal shear-to-flexure strength ratio is defined as $V_{ne}/V_{@Mne}$, where V_{ne} is the nominal shear strength of the wall and $V_{@Mne}$ is the shear demand at nominal moment, using expected material properties in both. For tests, this ratio is computed for the tested materials, whereas for the damaged building, this ratio is computed for expected material properties.

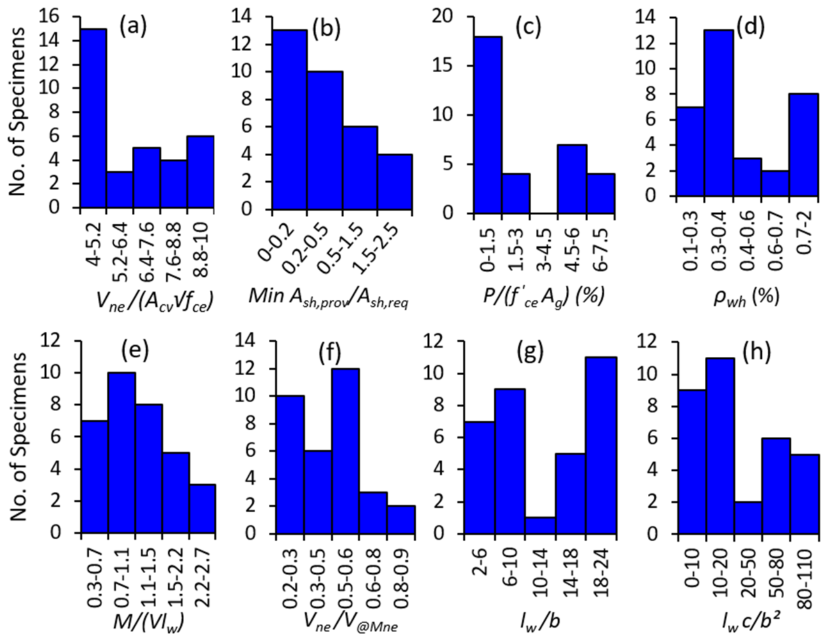


Figure B-9 Histograms of the shear-controlled DT/DC walls VDS database.

B.4.5 Shear-Friction-Controlled Walls

B.4.5.1 DESCRIPTION OF DATABASE

A database of 25 shear-controlled wall tests with shear-friction failure was assembled from a database of 71 shear wall tests. Histograms for several dataset parameters are shown in Figure B-10. Most of the specimens are walls with small ratios of base moment-to-base shear and horizontal web reinforcement ratios less than 1.1%, as shown in Figure B-10 (e) and (d), respectively.

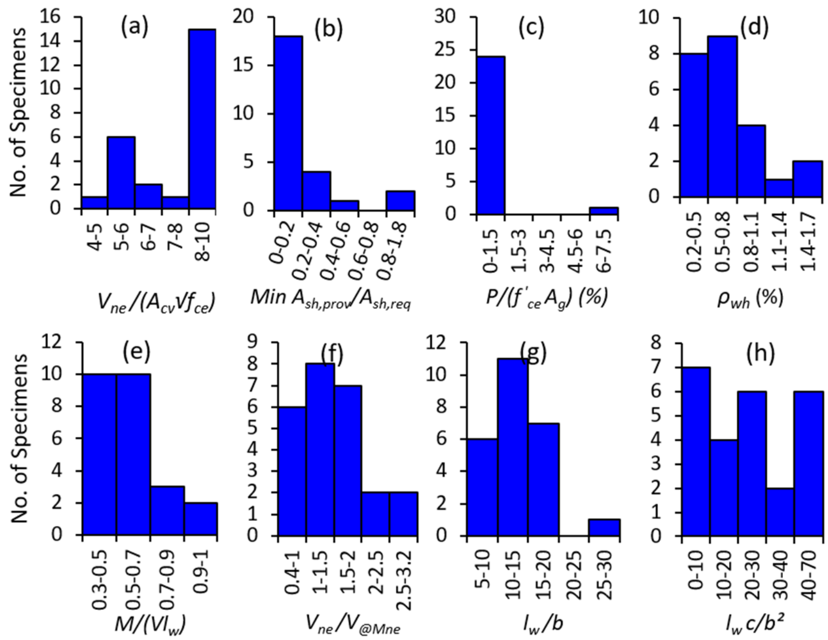


Figure B-10 Histograms of the shear-friction-controlled walls VDS database.

B.4.5.2 GUIDANCE ON IDENTIFYING REPRESENTATIVE TESTS

The key parameters to identify representative tests are the nominal friction-shear-to-flexure strength ratio, the nominal shear-to-flexure strength ratio and the slenderness parameter. The nominal friction-shear-to-flexure strength ratio is defined as $V_{nfe}/V_{@Mne}$, where V_{nfe} is the nominal shear-friction strength of the wall using expected material properties and $V_{@Mne}$ is the shear demand at nominal moment using expectep material properties. The nominal shear-to-flexure strength ratio is defined as $V_{ne}/V_{@Mne}$, where V_n is the nominal shear strength of the wall using expected material properties. The slenderness parameter is described in detail in Section B.4.1.2.

B.4.6 Coupling Beams

B.4.6.1 DESCRIPTION OF DATABASE

A database of 11 conventionally reinforced and 9 diagonally reinforced coupling beam tests was assembled from a database of 111 tests. Histograms for some dataset parameters are shown in Figure B-11 and in Figure B-12 for conventionally and diagonally reinforced coupling beams, respectively, where f'_{ce} is the expected (or measured) concrete compressive strength, and l_n/h is the beam aspect ratio (l_n is the clear span of the coupling beam and h is the total height). Figure B-11 (d) shows the cross-section shape of the conventionally reinforced specimens. Most of the conventionally reinforced specimens are coupling beams with an aspect ratio larger than 2 and a concrete compressive strength higher than 7.25 ksi (50 MPa), as shown in Figure B-11 (b) and (c), respectively. Most of the diagonally reinforced specimens are beams with an aspect ratio smaller than 2 as shown in Figure B-12 (b).

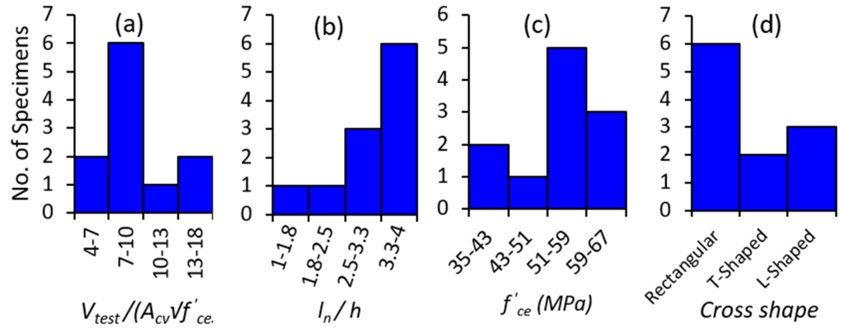


Figure B-11 Histograms of the conventionally reinforced coupling beams VDS database.

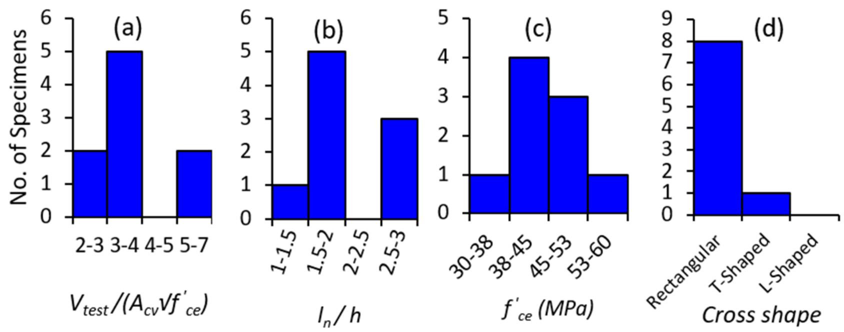


Figure B-12 Histograms of the diagonally reinforced coupling beams VDS database.

B.4.6.2 GUIDANCE ON IDENTIFYING REPRESENTATIVE TESTS

The key parameters to identify representative tests are the type of reinforcement (diagonal versus longitudinal) and the aspect ratio (l_n/h). Reinforcement of the coupling beam is either conventional or diagonal.

B.4.7 Slab-Column Connections

B.4.7.1 DESCRIPTION OF DATABASE

A database of 18 reinforced concrete and four post-tensioned slab-column connection tests with no shear reinforcement was assembled from a database of 121 tests. Histograms for the main parameters are shown in Figure B-13. The gravity shear ratio is defined as $V_g/\phi V_o$, where V_g is the gravity force to be transferred from the slab to the column, ϕ is the strength reduction factor (assumed equal to 1.0), and V_o is the punching shear strength calculated in accordance with ACI 318. Continuity reinforcement for slab-column connections in ASCE/SEI 41 and ACI 369 is defined as the presence of bottom slab bars passing through the column in each direction with $A_{s,min} \geq 0.5V_g/(\phi f_{ye})$. However, in ACI 318, continuity reinforcement (structural integrity) is defined as the presence of at least two of the column strip bottom bars in each direction passing within the region bounded by the longitudinal reinforcement of the column. Compliance of the specimens with these requirements is shown in Figure B-13 (b) and (c).

B.4.7.2 GUIDANCE ON IDENTIFYING REPRESENTATIVE TESTS

The key parameters to identify representative tests are the gravity shear ratio, continuity reinforcement by ASCE/SEI 41, continuity reinforcement by ACI 318, and type of reinforcement in slab (reinforced concrete versus post-tensioned).

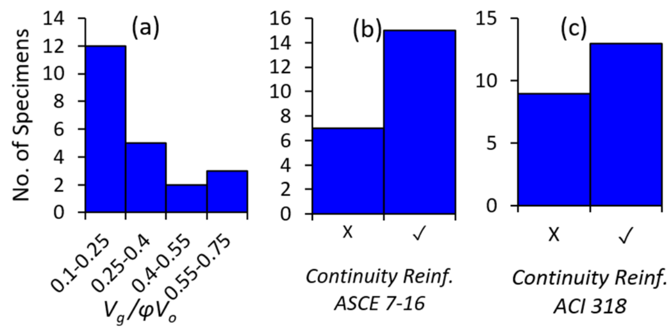


Figure B-13 Histograms of the slab-column database.

B.4.8 Nonconforming Shear-Controlled Columns

A database of 16 nonconforming shear-controlled columns was collected. Columns in this database fail along a diagonal plane, yielding of flexural reinforcement is not observed, and the measured peak strength is lower than required to reach the calculated flexural strength.

The key information reported in the database (and all frame component databases) are the member cross-section dimensions, axial load ratio ($P/A_g f'_{cd}$), aspect ratio (L_c/d), stirrup spacing to effective depth ratio (s/d), transverse reinforcement ratio (ρ_t), longitudinal reinforcement ratio (ρ_L), concrete compressive strength (f'_{ce}), expected flexural strength (V_p), expected undegraded shear capacity (V_o) defined according to the ASCE/SEI 41, and shear capacity ratio (V_p/V_o). Histograms for the main parameters are shown in Figure B-14.

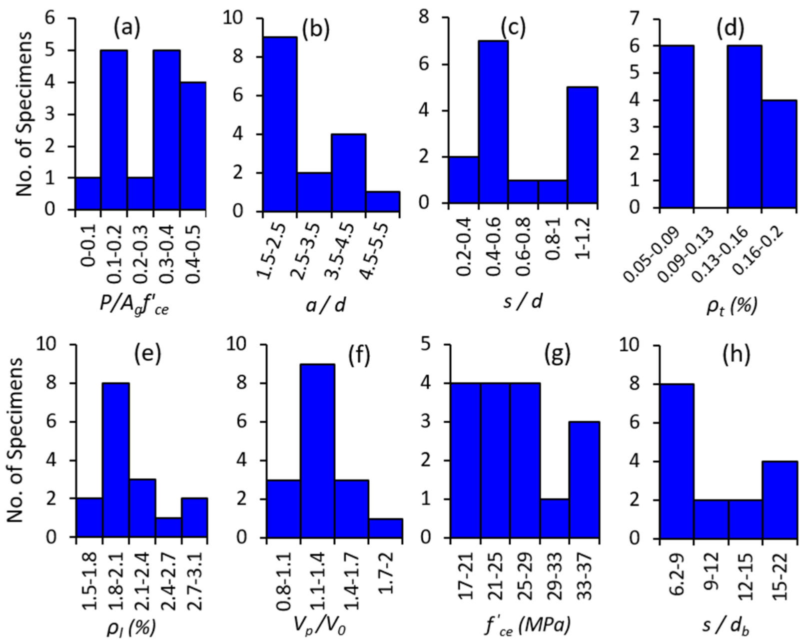


Figure B-14 Histograms of the shear-controlled columns database, where *a/d* represents aspect ratio or *L_c/d*.

B.4.8.1 GUIDANCE ON IDENTIFYING REPRESENTATIVE TESTS

Key parameters to be considered when identifying representative test specimens from the database include shear capacity ratio (V_p/V_o), transverse reinforcement ratio (ρ_t), and axial load ratio ($P/A_g f'_{ce}$).

B.4.9 Nonconforming Flexure-Shear-Controlled Columns

A database of 22 flexure-shear controlled columns was assembled. Tests in this database include columns where flexural yielding is reported and the measured peak strength is greater than that required to reach the calculated flexural strength.

The key information reported in the database for each column specimen is the same for all frame elements databases and is detailed in Section B.4.8. Histograms for the main parameters are shown in Figure B-15.

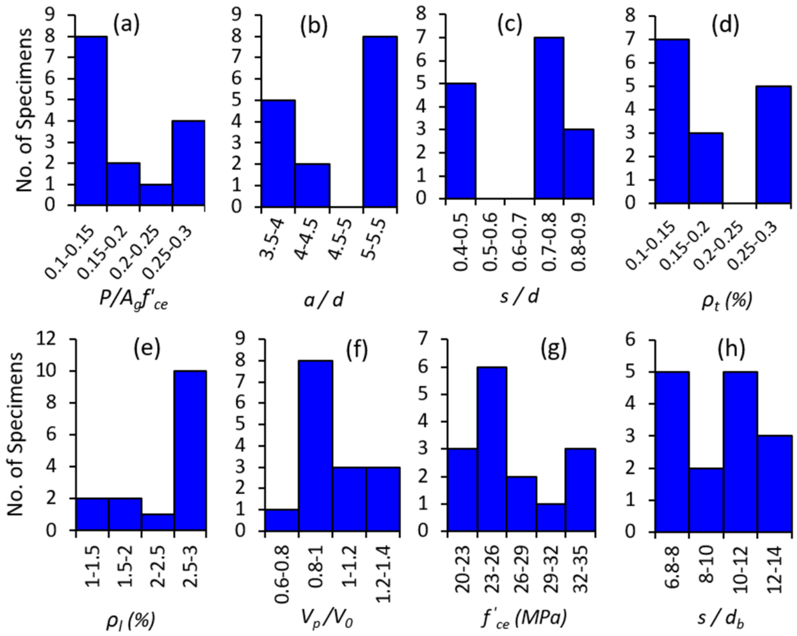


Figure B-15 **Histograms of the flexure-shear-controlled columns database, where *a/d* represents aspect ratio or L_c/d.**

B.4.9.1 GUIDANCE ON IDENTIFYING REPRESENTATIVE TESTS

Key parameters to identify representative test specimens from the database include shear capacity ratio (V_p/V_o), transverse reinforcement ratio (ρ_t), and axial load ratio ($P/A_g f'_{ce}$).

B.4.10 Conforming Ductile Columns

A database of 20 conforming columns was assembled. Key parameters are the same for all frame elements databases and is detailed in Section B.4.8. Histograms for the main parameters are shown in Figure B-16.

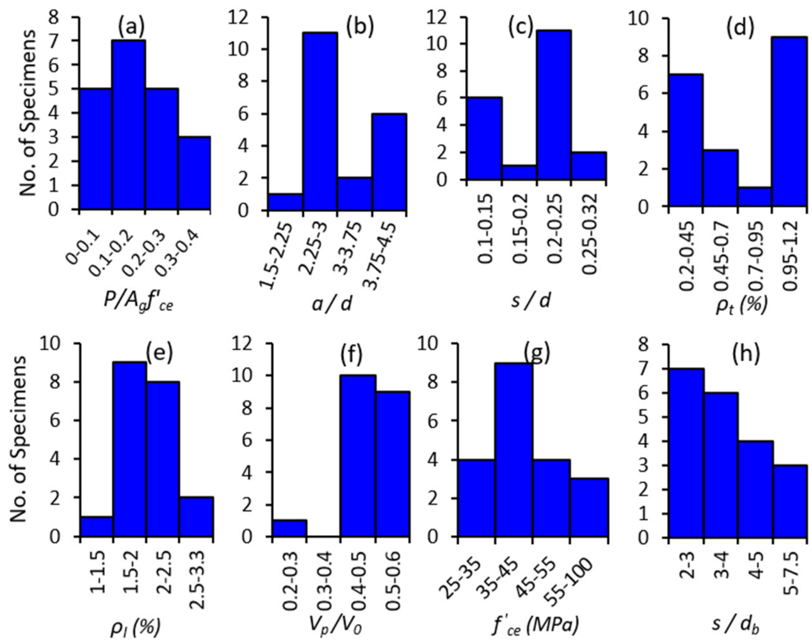


Figure B-16 **Histograms of the ductile columns database, where *a/d* represents aspect ratio or *L_c/d*.**

B.4.10.1 GUIDANCE ON IDENTIFYING REPRESENTATIVE TESTS

Key parameters to identify representative test specimens from the database include longitudinal reinforcement ratio (ρ_l), axial load ratio ($P/A_g f'_{ce}$), and transverse reinforcement ratio (ρ_t).

B.4.11 Lap-Splice-Controlled Columns

A database of eight lap-splice-controlled columns was collected. Columns in this database fail along the lap splice, yielding of flexural reinforcement may be observed, and the measured peak strength is lower than required to reach the calculated flexural strength.

The key information reported in the database are the member cross-section dimensions, axial load ratio ($P/A_g f'_{ce}$), aspect ratio (a/d), stirrup spacing to effective depth ratio (s/d), transverse reinforcement ratio (ρ_t), longitudinal reinforcement ratio (ρ_L), expected concrete compressive strength (f'_{ce}), expected flexural strength (V_p), expected undegraded shear capacity (V_0) defined according to the ASCE/SEI 41, and shear capacity ratio (V_p/V_0). Additionally, the lap splice length provided (l_{splice}), the confining force ratio along the splice length ($A_{vsp} f_{yet}/A_{sl} f_{yel}$), and the splitting distance of bar to longitudinal bar diameter ratio (c_b/d_b) are given. Histograms for the main parameters are shown in Figure B-17.

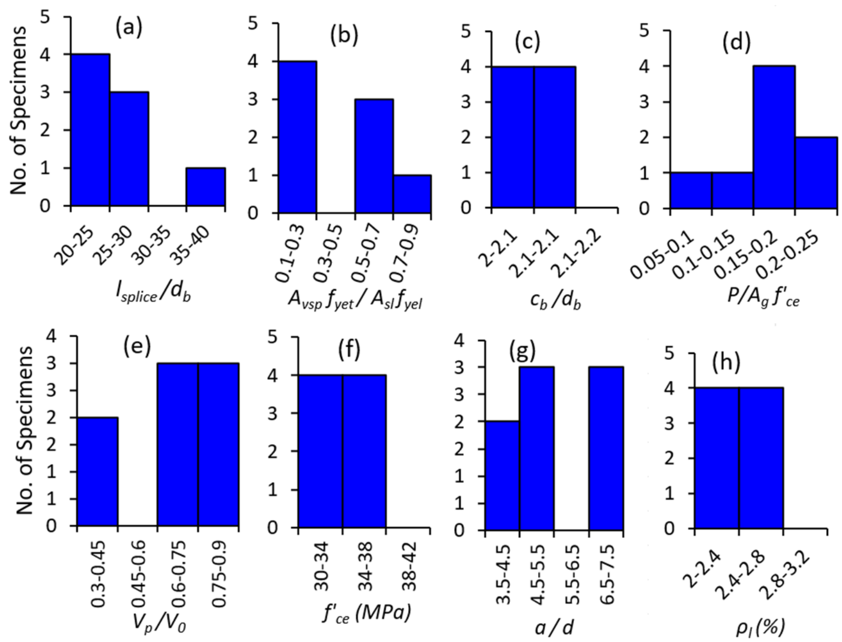


Figure B-17 Histograms of the lap-splice-controlled columns database.

B.4.11.1 GUIDANCE ON IDENTIFYING REPRESENTATIVE TESTS

Key parameters to be considered when identifying representative test specimens from the database include shear capacity ratio (V_p/V_o), transverse reinforcement ratio (ρ_t), and axial load ratio ($P/A_g f'_c$).

B.4.12 Conforming Ductile Beams

A database of 23 ductile beams was assembled. The key information reported in the database is the same for all frame elements databases and is detailed in Section B.4.8, except for the axial load ratio, which does not apply. Histograms for the main parameters are shown in Figure B-18.

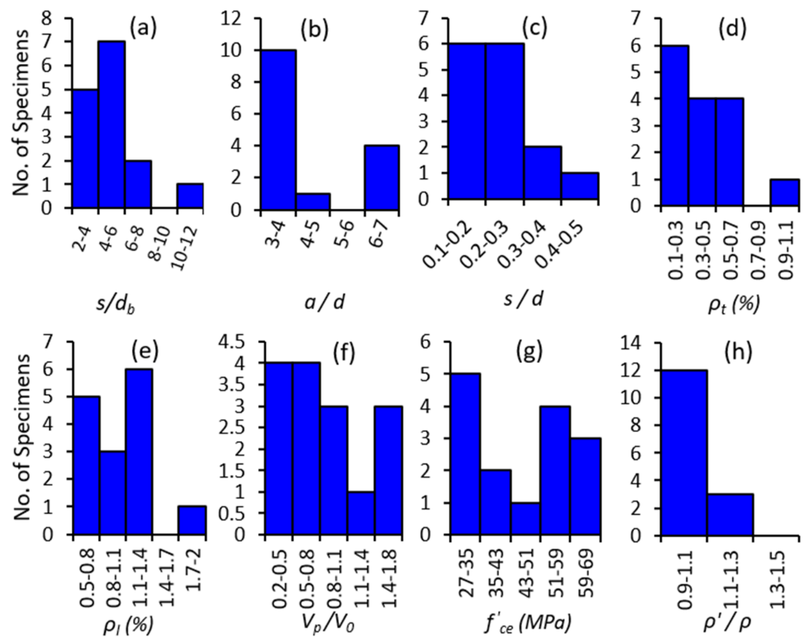


Figure B-18 Histograms of the ductile beams database.

B.4.12.1 GUIDANCE ON IDENTIFYING REPRESENTATIVE TESTS

Key parameters to use when identifying representative test specimens from the database include reinforcement ratio (ρ), aspect ratio (L_c/d), and transverse reinforcement ratio (ρ_t).

B.4.13 Interior Beam-Column Joints: Beam-Yielding/Joint-Failure Controlled

A database of 15 interior beam-column joints with joint failure after beam yielding was assembled using information from the database of reinforced concrete beam-column connections tests reported by Kim and LaFave (2007). Key parameters for each test include the beam and column geometry, column axial load ratio ($P/A_g f'_{ce}$), joint aspect ratio (h_b/h_c), joint reinforcement details, transverse and longitudinal reinforcement in the columns and beams, concrete compressive strength (f'_{ce}), nominal joint shear capacity (V_{nj}) defined according to the ASCE/SEI 41, expected shear demand at yielding of beam longitudinal reinforcement using f_s = 1.25f_y ($V_{@Mpb}$), and the shear capacity ratio ($V_{@Mpb}/V_{nj}$). Histograms for the main parameters are shown in Figure B-19.

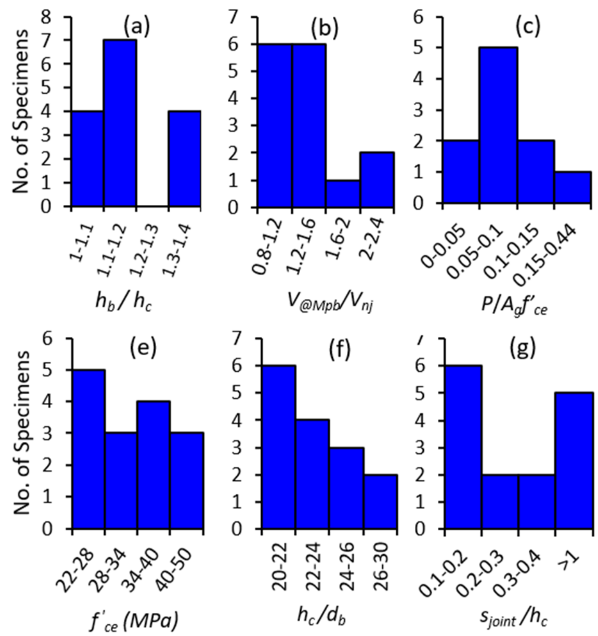


Figure B-19 Histograms of the BJ interior joint database.

B.4.13.1 GUIDANCE ON IDENTIFYING REPRESENTATIVE TESTS

Key parameters to consider when identifying representative test specimens from the database include column axial load ratio ($P/A_g f'_{ce}$), joint hoop spacing to joint width ratio (s_{joint}/h_c), and shear capacity ratio ($V_{@Mpb}/V_{nj}$).

B.4.14 Exterior Beam-Column Joints: Joint-Failure Controlled

A database of 7 exterior beam-column joints, with joint failure and without transverse reinforcement, was collected, using information from the database of reinforced concrete beam-column connections tests reported in the literature (Kim and LaFave, 2007). The key information reported in the database for each specimen are the joint, beam and column geometry, axial load ratio ($P/A_g f'_{ce}$), joint aspect ratio (h_b/h_c), joint reinforcement details, transverse and longitudinal reinforcement in columns and beams, concrete compressive strength (f'_{ce}), expected joint shear strength coefficient (V_{nj}), expected shear demand at yielding of beam longitudinal reinforcement using $f_s = 1.25f_y$ ($V_{@Mpb}$), and the shear capacity ratio ($V_{@Mpb}/V_{nj}$). Histograms for the main parameters are shown in Figure B-20.

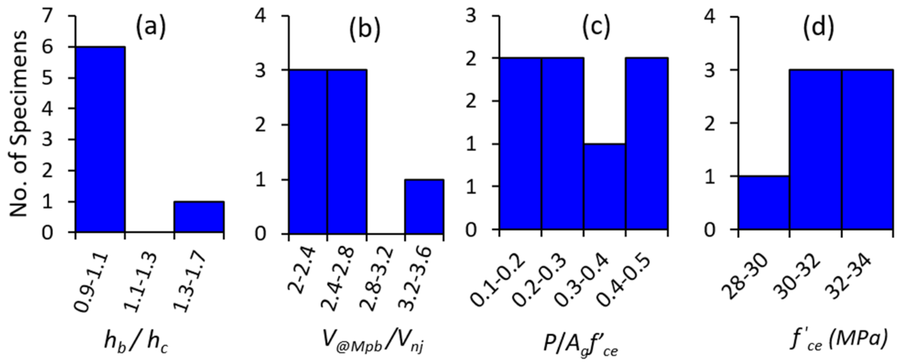


Figure B-20 Histograms of the JF joint database.

B.4.14.1 GUIDANCE ON IDENTIFYING REPRESENTATIVE TESTS

Key parameters to consider when identifying representative test specimens from the database include column axial load ratio ($P/A_g f'_{ce}$), and joint aspect ratio (h_b/h_c).

Appendix C: Fatigue Capacity Models and Background

C.1 Overview

This appendix provides background to the recommendations made in Section 5.6 of the *Guidelines* regarding assessment of the impact of low-cycle fatigue (LCF) on the future performance of earthquake-damaged reinforced concrete structures. It also discusses the methods that may be used where further fatigue checks are required for specific elements.

Additional information can be found in Appendix C of *Resilient Repair Guide Source Report: Post-Earthquake Assessment of Reinforced Concrete Buildings* (ATC, 2021a).

C.2 Basis of Fatigue Screening Check

The residual fatigue life of reinforcement in an earthquake-damaged component or structure is considered sufficient if either:

1. The fatigue demands imposed on the reinforcement by the damaging earthquake were negligible, or
2. The impacted reinforcement is able to withstand the demands of a future risk-targeted Maximum Considered Earthquake (MCE_R) without fracture.

Fulfillment of either of these criteria is considered sufficient to demonstrate that the reinforcement has not been compromised by fatigue damage significantly enough to impact the building performance in future earthquakes. It is not necessary for a structure to fulfill both.

Considering both criteria above, the level of fatigue damage considered to be acceptable during a damaging earthquake is as shown in Figure C-1. The level of acceptable damage varies depending on the intensity of the damaging earthquake, represented as the ratio of spectral acceleration at 1 second, S_{a1}, for the damaging earthquake to S_{a1} for an MCE_R at the site. This figure is defined on the following basis:

- The acceptable fatigue damage ratio during a low intensity earthquake is set as 10%. This represents a level of fatigue degradation that has no more than a minor impact on the risk of fatigue failure during a future earthquake. Such a threshold is subjective, and there is no specific research that justifies the value of 10%. However, a similar threshold has been proposed on occasions as 'de minimis' by building owners' engineers during discussions of earthquake damage insurance claims in New Zealand, where the required standard of repair is generally more onerous than the assurance of future safety of a building, which is the focus here.

- The acceptable damage in a structure that has already been subjected to MCE_R shaking is set as 50%. This leaves a residual fatigue capacity of 50%, which would logically be sufficient to just accommodate further MCE_R shaking.

- Between these two values, the permitted fatigue damage is linearly interpolated.

- For long duration earthquake shaking (i.e. significant duration, $D_{5\text{-}95}$, greater than 45 seconds), where there is a greater likelihood for low-cycle fatigue damage, it is considered prudent to remain below the 10% limit regardless of the intensity of the damaging earthquake. $D_{5\text{-}95}$ is a measure of the time it takes for an earthquake to accumulate the central 90% of its total energy.

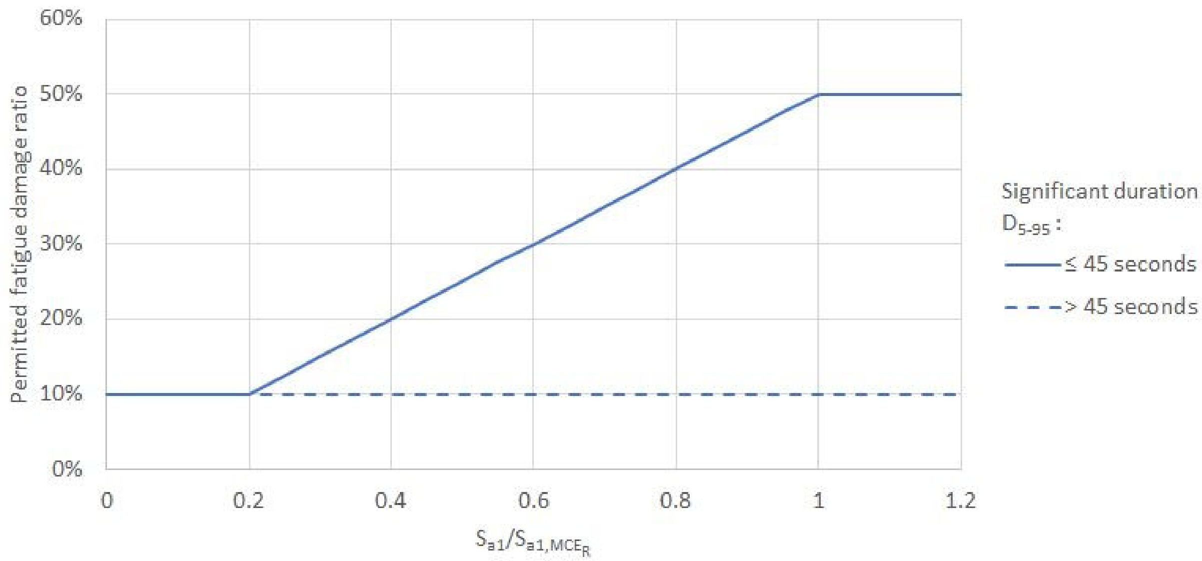


Figure C-1 Relationship between acceptable fatigue damage and shaking intensity of the damaging earthquake.

The acceptable fatigue damage levels shown in Figure C-1 form the basis for the fatigue screening checks described in Section 5.6. All screening checks were developed using the assumption that the cyclic demand imposed by an earthquake with significant duration ($D_{5\text{-}95}$) less than 45 seconds can be represented by a simplified displacement history of the type specified by FEMA 461, *Interim Testing Protocols for Determining the Seismic Performance Characteristics of Structural and Non-Structural Components* (FEMA, 2007), but with three reversed cycles per displacement increment, which corresponds to 6.1 effective cycles to the peak displacement as described in Appendix C of ATC (2021a). Using this basis, the checks outlined in Section 5.6 were derived as follows:

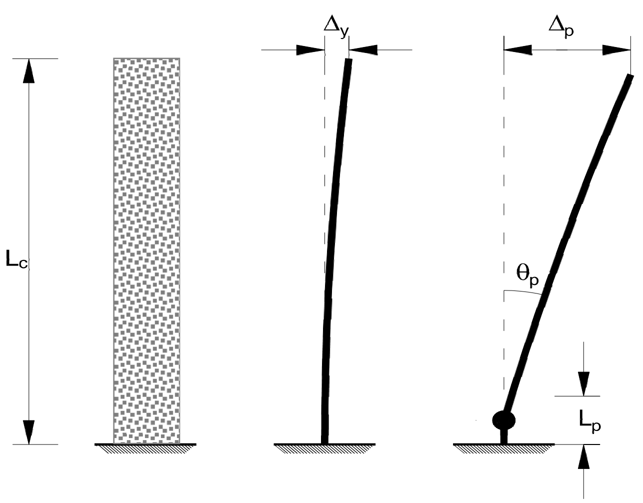
- The reinforcement strain limits given in Figure 5-17 were derived using the coefficients proposed by Marder (2018) for the Koh and Stephens (1991) fatigue life equation to determine the tension strain corresponding to the permitted fatigue damage ratio. For each permitted damage ratio, the damage per effective cycle was calculated along with the corresponding number of cycles to failure. The cycles to failure then permit determination of the acceptable strain amplitude (ε_a) and hence the tension strain limit, with the latter being determined assuming that the compression strain was equal to 15% of the tension strain.

- The beam screening check was developed to check whether 0.02 radian chord rotation could be sustained without exceeding 10% fatigue damage. The full basis for the beam screening check is outlined in ATC (2021a).

- The wall screening check was developed by determining the plastic rotation corresponding to a tensile strain of 0.03 or less.

C.3 Estimation of Maximum Reinforcement Tensile Strain

In order to undertake the strain-based screening fatigue check, it is necessary to estimate the maximum reinforcement tensile strain during the damaging earthquake. For the common case of elements governed by flexural yielding this can be achieved based on the equivalent cantilever shown in Figure C-2.



(a) Member **(b) Elastic deformation** **(c) Inelastic deformation**

Figure C-2 Idealized equivalent cantilever, elastic, and plastic components of deflection

In order to determine the reinforcement strain, it is necessary to estimate the plastic portion of the rotation demand, θ_p. This can be calculated as:

$$\theta_p = \Delta_p / \left(L_c - L_p / 2\right) \tag{C-1}$$

where Δ_p is the plastic displacement, which can be calculated as the total displacement minus the yield displacement, Δ_y. The yield displacement can be determined by any rational procedure, for example those used to calculate the displacement associated with point “B” in generalized force-deformation relationships defined in ASCE/SEI 41. Alternatively, and conservatively, it may be assumed that the entire deflection arises due to plastic rotation.

The maximum reinforcement strain can then be estimated as:

$$\varepsilon_s = \varepsilon_y + \theta_p \left(h - c\right) / L_p \quad \text{(C-2)}$$

where:

ε_y = reinforcement yield strain

L_p = effective plastic hinge length, which can be estimated based on Equations C-3 or C-4 below

h = depth of section

c = depth to neutral axis

For beams and columns, the plastic hinge length can be estimated as:

$$L_p = k_{lp}L_c + L_{sp} \geq 2L_{sp} \quad \text{(C-3)}$$

where:

k_{lp} $= 0.2\left(\frac{f_u}{f_{ye}} - 1\right) \leq 0.08$ unless the longitudinal reinforcement ratio is less than the minimum required by ACI 318, in which case $k_{lp} = 0$

L_c = shear span, i.e., the distance of the critical section from the point of contraflexure

L_{sp} = strain penetration length $= 0.15f_{ye}d_b$ where the units of f_{ye} are ksi, or $0.022f_{ye}d_b$ if the units of f_{ye} are MPa

f_{ye} = expected yield strength of longitudinal reinforcement

d_b = diameter of longitudinal reinforcement

f_u = expected ultimate strength of the longitudinal reinforcement

For walls with longitudinal reinforcement that exceeds the minimum ratio required by ACI 318, the plastic hinge length can be estimated as:

$$L_P = k_{lp}L_c + 0.1l_w + L_{sp} \quad \text{(C-4.a)}$$

where l_w is the wall length and other terms are as defined for Equation C-3.

For walls where the longitudinal reinforcement is less than the minimum required by ACI 318, the plastic hinge length should be estimated as:

$$L_P = 2L_{sp} \tag{C-4.b}$$

Strain demands can also be extracted directly from nonlinear fiber elements used by some nonlinear analysis software. Such strains can be very sensitive to nonlinear modeling assumptions, particularly for force-based fiber elements, and should be used with caution.

C.4 Further Fatigue Check

If reinforcement fails the screening check described in the preceding section, more detailed checks on reinforcement condition can be undertaken.

Further fatigue checks can be undertaken in two ways, namely:

- By a simplified fatigue life assessment using artificial displacement histories to represent the demands previously imposed by the damaging earthquake, and likely to be imposed by MCE_R shaking, or
- By use of a detailed fatigue damage assessment methodology, for example rainflow counting, to determine a fatigue damage sum based on strain demands obtained from response history analysis.

C.4.1 Simplified Fatigue Life Assessment

Calculation of fatigue damage requires not only an estimate of the peak deformation imposed on a structural element, but also an estimate of the complete deformation history (i.e., cyclic response) imposed during a damaging earthquake. As elaborated on in Section C4.2, detailed analysis to determine this deformation history is complex and influenced by the specific characteristics of the structure and the damaging earthquake.

In lieu of such an approach, a simplified method of estimating fatigue damage has been developed for this study. The simplified approach may be applied to determine whether fatigue damage is problematic for cases that exceed the thresholds established in this report.

The premise of the simplified fatigue life assessment is that the cyclic deformation imposed by an earthquake may be satisfactorily approximated by consideration of an artificial displacement history of the type described in FEMA 461 for quasi-static cyclic testing. The steps involved include:

1. Derive displacement history for the damaging earthquake, and, if necessary, future MCE_R shaking.

2. Calculate total or plastic strain amplitude for each displacement increment.
3. Determine fatigue damage corresponding to each displacement increment using the recommended fatigue life relationships.
4. Calculate the sum of the fatigue damage for the damaging earthquake, and, if required, for future MCE_R shaking.
5. The remaining fatigue life is considered sufficient if:
    a. The fatigue damage from the damaging earthquake is less than the amount shown in Figure C-1, or
    b. The remaining fatigue life is sufficient to sustain a future MCE_R event.

These steps are discussed in more detail below.

C.4.1.1 ESTABLISH DISPLACEMENT HISTORY

The displacement history deemed appropriate for either the damaging earthquake, or a future MCE_R, is based on the FEMA 461 displacement history for quasi-static cyclic testing. For the simplified fatigue assessment this is defined as follows:

- The displacement history is to comprise a series of increments of increasing drift amplitude, with the first increment being less than the yield drift and successive increments being 1.4 times larger than the preceding increment.
- The maximum drift should be the drift estimated to have occurred during the damaging earthquake, or for the MCE_R, the drift expected to occur.
- For earthquakes with significant duration ($D_{5\text{-}95}$) not exceeding 45 seconds, three reversed cycles are applied at each drift increment. For longer duration earthquakes, specific study would be required to determine the appropriate number of cycles.

An example of this approach is shown in Figure C-3, where the drift during the damaging earthquake is estimated as 1.5%.

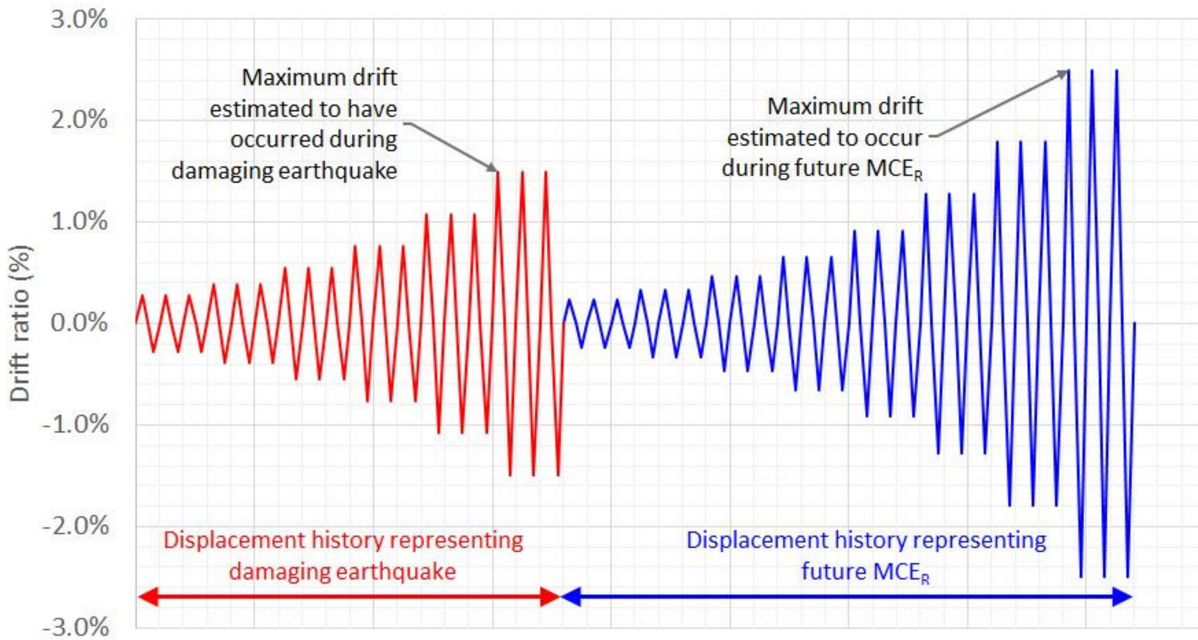


Figure C-3 Example displacement history where fatigue assessment considers the impact of a damaging earthquake estimated to have caused a maximum drift of 1.5% and a future MCE$_R$ expected to cause a maximum drift of 2.5%.

C.4.1.2 STRAIN AMPLITUDE

As outlined in the following section, fatigue damage is generally calculated based on either the total strain amplitude or the plastic strain amplitude. The choice to use total strain amplitude or plastic strain amplitude as the basis of fatigue calculations can be made by the engineer based on whichever is most convenient.

The strain amplitude, ε_a, is defined as half the total strain range (i.e., most tensile to most compressive) experienced by a bar. The total strain amplitude can therefore be calculated as:

$$\varepsilon_a = \frac{\varepsilon_t - \varepsilon_c}{2} \tag{C-5}$$

where:

ε_t = tension strain

ε_c = compression strain, with compressive strains negative.

The plastic strain amplitude, ε_{ap}, is then given by:

$$\varepsilon_{ap} = \varepsilon_a - \varepsilon_y \qquad \text{(C-6)}$$

where:

ε_y = reinforcement yield strain

C.4.1.3 FATIGUE DAMAGE PER DISPLACEMENT INCREMENT

Materially less fatigue damage occurs when reinforcement is supported by closely spaced transverse reinforcement that reduces the occurrence of reinforcement buckling. Reinforcement buckling significantly reduces fatigue life because it induces large local plastic strains and can lead to cracking at reinforcement deformations (Restrepo-Posada, 1993) that in turn leads to failure of the bar. The fatigue checks required by Section 5.6 are for bars that have not visibly buckled (visibly buckled bars are assumed to require repair and hence do not need to be assessed for fatigue damage). Therefore, the recommended fatigue damage assessment equations below only consider bars that are unlikely to buckle by using data to determine empirical coefficients from specimens with transverse reinforcement spacing of $4d_b$ or less.

The number of half cycles ($2N_f$) that can be sustained by a reinforcing bar before failure is dependent on the strain amplitude imposed during each cycle. For any particular strain level, *i*, the damage imposed by a single half cycle can be calculated as:

$$d_i = 1/(2N_{f(i)}) \qquad \text{(C-7)}$$

where:

$2N_{f(i)}$ = number of half cycles to failure that can be sustained for the strain imposed during cycle *i*.

The relationship between half cycles to failure and imposed strain is generally either given in the form of:

- the Coffin-Manson equation (Coffin, 1953; Manson, 1954) that relates plastic strain amplitude, ε_{ap}, to the number of half cycles required to cause fracture, $2N_f$, or
- the Koh and Stephens (1991) equation that relates total strain amplitude, ε_a, to the number of half cycles required to cause fracture.

The Coffin-Manson equation is defined as:

$$\varepsilon_{ap} = \varepsilon_f^{'}\left(2N_f\right)^c \tag{C-8}$$

where:

ε_{ap} = plastic strain amplitude

$\varepsilon_f^{'}$ = an empirical coefficient = 0.12

$2N_f$ = number of half-cycles to failure

c = an empirical coefficient = -0.31

The above recommended empirical coefficients are based on Zhong and Deierlein (2019) assuming $s/d_b = 4$, $d_b = 25$ mm, $f_y = 420$ MPa, $T/Y = 1.4$, $E_s = 200{,}000$ MPa.

The Koh and Stephens equation has a similar form, being:

$$\varepsilon_a = M\left(2N_f\right)^m \tag{C-9}$$

where:

ε_a = total strain amplitude

M = an empirical coefficient = 0.09

m = an empirical coefficient = -0.41

The above recommended empirical coefficients are based on Marder (2018).

C.4.1.4 FATIGUE DAMAGE SUM

Following calculation for each drift increment of the fatigue damage caused by a single half cycle, d_i, the total fatigue damage expected to have occurred during the damaging earthquake or the MCE_R can be calculated using Miner's rule (Miner, 1945), which can be formulated as:

$$D = n\sum d_i = \sum {}^{n}\!\!\diagup\!_{\left(2N_{f(i)}\right)} \tag{C-10}$$

where:

D = a damage index where a value of 1.0 is generally linked to incipient fracture

n = number of half cycles assumed to have occurred to each drift increment, i.e., 3 for the typical case as outlined in Section C.4.1.1

The residual capacity of a reinforcing bar can be considered acceptable if, following calculation of damage sums for the damaging earthquake and, if required, the MCE_R, either:

- The damage sum from the damaging earthquake is less than the limit shown in Figure C-1, or
- The total damage sum from the damaging earthquake and the future MCE_R are less than 1.0.

If neither of the above conditions is fulfilled, then the bar has sustained excessive fatigue damage and reinforcement repair is required. Section 5.9.5 on repair techniques for steel reinforcement should be consulted.

C.4.2 Recommendations for Detailed Fatigue Life Assessments

Detailed assessment of fatigue damage is a complex process. Only limited comments on the recommended approach are given here; interested readers are referred to Zhong and Deierlein (2019) for further details.

The aim of detailed assessment of fatigue damage should be to obtain the best possible estimate of the strain history sustained by the reinforcing bars considered. This necessitates use of nonlinear response history analysis, with the input ground motion used for the analysis being the best available estimate of shaking at the site of the damaged building. If multiple, representative records of nearby ground shaking are available, it is recommended that multiple analyses be undertaken. The results should be averaged unless comparison to other observable damage suggests that one particular record is more appropriate. Refer to Section 3.4.3 for more details on identification of appropriate ground motions.

Where response history analysis is used to verify that the residual fatigue capacity of reinforcement is sufficient to withstand the MCE_R for the site (performance measure 2 as defined in Section C.2), ground motions representative of the MCE_R should be chosen in accordance with appropriate guidance for new building design in the same jurisdiction.

Following completion of the response history analyses, fatigue damage should be determined based on either the Coffin-Manson or Koh and Stephens equations as specified in Section C.4.1.3. Damage summation should be completed using Miner's sum, as described in Section C4.1.4, or other appropriate summation method, with effective cycle strain magnitudes calculated following a rainflow counting approach (Zhong and Deierlein, 2019).

The residual capacity of a reinforcing bar can be considered acceptable if, following calculation of damage sums for the damaging earthquake and, if required, the MCE_R, either:

- The damage sum from the damaging earthquake is less than the limit shown in Figure C-1, or
- The total damage sum from the damaging earthquake and the future MCE_R are less than 1.0.

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Appendix D: Relation to the IEBC

D.1 Context

These *Guidelines* are aligned with the IEBC. Specific concepts addressed in the IEBC and integrated herein include substantial structural damage and disproportionate earthquake damage. This appendix summarizes the relation between these concepts in the IEBC and the concepts as they appear in these *Guidelines*. The reference document, unless otherwise noted, is the 2024 IEBC.

D.2 Substantial Structural Damage

D.2.1 As Defined in the IEBC

According to the IEBC, substantial structural damage is a condition where any of the following apply:

1. The vertical elements of the lateral-force-resisting system have suffered damage such that the lateral-load-carrying capacity of any story in any horizontal direction has been reduced by more than 33% from its pre-damaged condition.
2. The capacity of any vertical component carrying gravity load, or any group of such components, that has tributary area more than 30% of the total area of the structure's floor(s) and roof(s) has been reduced more than 20% from its pre-damaged condition, and the remaining capacity of such affected elements, with respect to all dead and live loads, is less than 75% of that required by the IBC for new buildings of similar structure, purpose, and location.
3. The capacity of any structural component carrying snow load, or any group of such components, that supports more than 30% of the roof area of similar construction has been reduced more than 20% from its pre-damaged condition, and the remaining capacity with respect to dead, live, and snow loads is less than 75% of that required by the IBC for new buildings of similar structure, purpose, and location.

A building that has sustained substantial structural damage to the vertical elements of the lateral-force-resisting system needs to be evaluated and either repaired or repaired and retrofitted depending on the results of the evaluation. This designation is used to identify buildings that, on the basis of their earthquake damage, may warrant retrofit.

The IEBC also defines concepts of substantial damage and substantial improvement, which relate to flood hazard provisions. They are not relevant here.

D.2.2 As Appears in These *Guidelines*

As described in Section 4.4.3, the concept of substantial structural damage is used in these *Guidelines* to identify buildings in need of further evaluation and, potentially, retrofit. The concept of substantial structural damage herein is consistent with the definition of substantial structural damage in the IEBC in that it consists of a check on loss of lateral-load-carrying capacity (Condition 1) and a check on damage to gravity-load-carrying capacity (Condition 2). Crucially, the details of the gravity-load-carrying capacity condition are modified such that they are appropriate for effects of seismic damage on loss of gravity-load-carrying capacity.

D.2.2.1 LATERAL-LOAD-CARRYING CONDITIONS FOR SUBSTANTIAL STRUCTURAL DAMAGE

Condition 1 of the IEBC definition of substantial structural damage appears in these *Guidelines* without modification. However, these *Guidelines* provide several clarifications. Firstly, the *Guidelines* describe how the reduction in lateral-load-carrying capacity can be determined using the damage assessments in this document. This approach is consistent with the IEBC, as the IEBC does not describe how the reduction in lateral-load-carrying capacity is determined. Specifically, Section 4.4.4 states that any components that are assigned to DC2 shall be assumed to have a reduced strength in the calculation of the lateral capacity of the damaged building. This reduced strength of components in DC2 should generally be taken as zero. The reduced strength can be determined from analysis if it is shown through nonlinear analysis that the deformation demands on the component in the damaging earthquake did not exceed the Collapse Prevention (CP) acceptance criteria for that component, as defined in ASCE/SEI 41. This guidance is intended to facilitate post-earthquake decision making by reducing uncertainty and variability in how reduction of lateral-load-carrying capacity is determined in the identification of substantial structural damage, explicitly linking the damage classifications herein to the determination of substantial structural damage. It is also informed by studies (described below) that investigated the appropriateness of the 33% strength loss criterion for substantial structural damage.

Guidance is also provided to define for the user of these *Guidelines* about what is meant by vertical elements and what to do if the lateral-force-resisting system is not well defined. The *Guidelines* recommend that diaphragms should be included in the assessment of the capacity of the lateral-force-resisting system for Condition 1, despite the qualifier “vertical” used in the IEBC. Floor diaphragm and associated elements (e.g., collectors, drag elements, wall-diaphragm interface) are essential to the lateral-force-resisting system, such that damage to these elements compromises the lateral-force-resisting system. Moreover, the *Guidelines* indicate that the calculation of reduction in lateral-load-carrying capacity should consider all elements that are part of the lateral-force-resisting system, or all structural elements in the building if there is no clearly defined lateral system.

In developing these *Guidelines*, several buildings were analyzed to evaluate the 33% limit associated with substantial structural damage Condition 1. In this evaluation, substantial structural damage was interpreted as occurring when future earthquake performance was impaired. Impaired future earthquake performance was quantified by an increase in drift demands for the damaged building relative to the undamaged building. This Performance-Critical Damage was identified as having occurred when the deformation demands in the damaging earthquake exceeded the deformation capacity (i.e., point of peak load) of a component or system (ATC, 2021a; Murray et al., 2022; Safiey et al., 2022). To relate this Performance-Critical Damage to substantial structural damage, analyses of the performance of a damaged building (described in ATC, 2021a) were used to explore how well the point of 33% reduction in lateral-load-carrying capacity defined in Condition 1 aligned with the occurrence of Performance-Critical Damage.

Figure D-1 illustrate the outcome of these analyses for a reinforced concrete special moment frame, in which reduction in lateral-load-carrying capacity was determined as defined in these *Guidelines*, i.e. components in DC2 are assumed to have a residual capacity of 0. The plot shows that substantial structural damage (i.e., analyses in which the damaging earthquake led to 33% or greater strength loss, indicated by the red diamonds) tended to occur in analyses where the story drift exceeded 3% drift. This 3% drift value is similar to the peak drift associated with Performance-Critical Damage in this structure as determined by Murray et al. (2022).

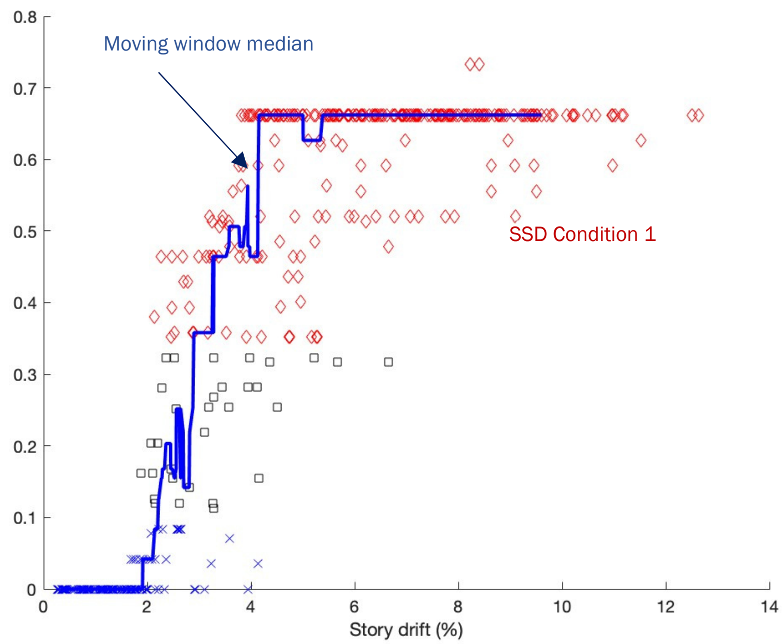

Figure D-1 Assessment of reduction in lateral-load-carrying capacity for a reinforced concrete special moment frame assessed for multiple ground motions and intensity levels.

However, these analyses also demonstrate that results are sensitive to how the reduction in lateral-load-carrying capacity is determined. Figure D-2 shows results for the same building but with the reduction in lateral-load-carrying capacity computed differently. In Figure D-2, damaged components were assumed to have lost some, but not all, of their strength. This analysis was conducted using nonlinear dynamic analyses followed by a pushover analysis of the damaged building, which was used to assess the reduction in capacity. As a result, the reductions in component stiffnesses and strengths are based on the hysteretic rules defined by Ibarra et al. (2005) and Haselton et al. (2016), and the damaged components have much more capacity than the assumption of 0 used in Figure D-1. Under these conditions, the reduction in lateral-load-carrying capacity past the 33% limit is not well aligned with the assessment of impaired performance and is unconservative; in other words, the analysis indicates that the building system has much impaired performance (i.e., amplified drifts on the y-axis) for damage that causes much less than 33% loss of lateral-load-carrying capacity. Consequently, the studies done herein indicate that the 33% loss of lateral capacity for definition of substantial structural damage is only well aligned with impaired performance if the damaged components are assigned a very small or 0 capacity in the calculation of reduced lateral-load-carrying capacity.

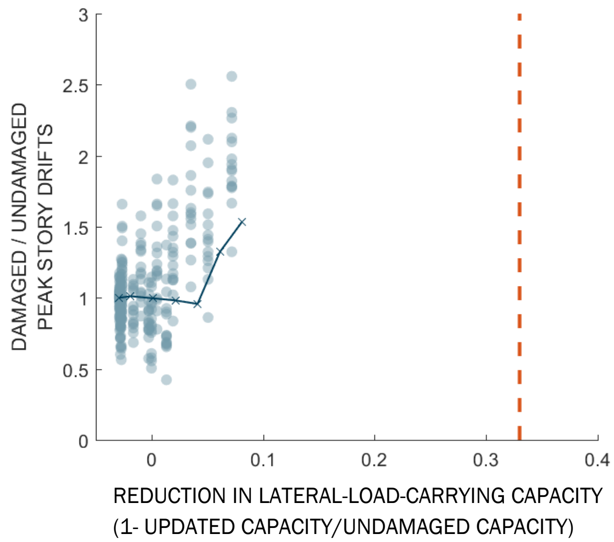

Figure D-2 Assessment of reduction in lateral-load-carrying capacity for a reinforced concrete ordinary moment frame from earthquake damage assessed for multiple ground motions and intensity levels.

D.2.2.2 GRAVITY-LOAD-CARRYING CONDITIONS FOR SUBSTANTIAL STRUCTURAL DAMAGE

The IEBC's Condition 2 (and 3) are intended to check the condition of vertical components carrying gravity (and snow) loads. However, it is not possible to robustly determine whether this condition exists for an earthquake-damaged building according to the IEBC provision because it requires the assessment of the loss of gravity capacity of earthquake-damaged components. Although this is an active area of experimental and analytical research, quantification of how much gravity capacity is lost when earthquake damage occurs is not well understood due to the potentially rapid and brittle progress of gravity failures. Therefore, Condition 2 for substantial structural damage in these *Guidelines* maintains the concept of checking the condition of gravity-load-carrying components but implements an alternative approach for this check that is consistent with available knowledge of earthquake damage.

The gravity-load-carrying capacity check in Condition 2 in these *Guidelines* consists of a list of gravity-essential components. Gravity-essential components are components whose failure compromises the ability of the structure to carry gravity loads. Such components, when classified as DC2, are vulnerable to a rapid loss of gravity-load capacity. As in IEBC Conditions 2 and 3, these criteria are used to evaluate components carrying more than 30% of the total area of the structure's floor(s) or roof(s). This approach is similar to that adopted by the San Francisco Administrative Bulletin AB-099 addressing evaluation of earthquake damage in nonductile reinforced concrete buildings (City of San Francisco, 2023).

The loads considered include gravity and snow loads where applicable.

D.3 Disproportionate Earthquake Damage

D.3.1 As Defined in the IEBC

According to the IEBC, disproportionate earthquake damage is a condition of earthquake-related damage where both of the following occur.

1. The 0.3-second spectral acceleration at the building site for the earthquake in question, as estimated by one of the following, is less than 30% of the mapped acceleration parameter S_s.
   - The USGS algorithm for the datapoint closest to the site, or
   - As determined from peer-reviewed seismograph records from the site or from a location closer to the site than the algorithm-provided datapoints.
2. The vertical elements of the lateral-force-resisting system have suffered damage such that the lateral-load-carrying capacity of any story in any horizontal direction has been reduced by more than 10% from its pre-earthquake condition.

Disproportionate earthquake damage only applies in Seismic Design Categories D, E, and F. According to the IEBC, disproportionate earthquake damage “exists where a building has significant damage in even a very small earthquake. This damage is an indicator of severe damage, possibly collapse, in a larger event.”

D.3.2 As Appears in These *Guidelines*

The same concept of disproportionate earthquake damage is used in these *Guidelines* to identify buildings in need of further evaluation and, potentially, retrofit.

Condition 1 in the definition of disproportionate earthquake damage appears in these *Guidelines* without modification except that any of the methods described for seismic demand representation in Section 3.4.3 may be used to determine the 0.3-second spectral acceleration. An example map is provided in Figure D-3, showing shaking intensities from the 1994 Northridge Earthquake. Most of the damage in this earthquake occurred in areas shaded yellow and red (i.e., where the damage would not be considered disproportionate).

Condition 2 in the definition of disproportionate earthquake damage appears in these *Guidelines* without modification. However, the same guidance applying inspection and analyses in these *Guidelines* to the determination of reduction of lateral-load-carrying capacity (see Section D.2.2) for substantial structural damage applies here.

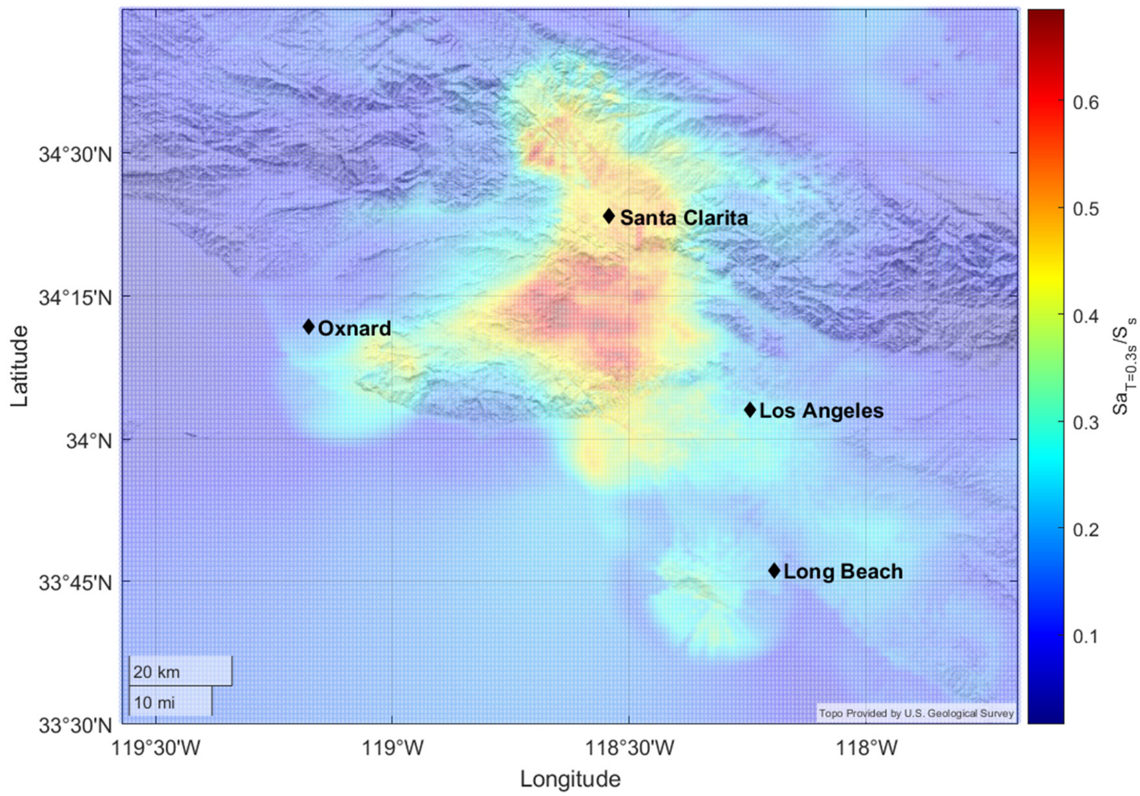

Figure D-3 Map of estimated shaking intensities from the USGS ShakeMap for the 1994 Northridge Earthquake, normalized by S_s. Any location where this ratio is less than 0.3 would be a possible location for disproportionate earthquake damage. For this illustration, the S_s values used are consistent with ASCE/SEI 7-16.

Glossary

The following definitions are provided to explain terms specific to assessment and repair of earthquake-damaged buildings as used in these *Guidelines*.

A

B

Building Repair Outcome. Determination as to whether or not the building requires performance-critical repairs or requires seismic retrofit in addition to performance-critical repairs (see Figure 4-1).

C

Compliant building. Building complying with acceptable codes and standards, as determined by one of the compliance methods described in the IEBC (see Section 4.4.1).

D

Damage Class (DC). Classification of the damage severity of the component (see Sections 2.2.4.2 and 4.3).

Damage State (DS). Key points on the component's cyclic envelope; DS2 is the critical Damage State and indicates the onset of Performance-Critical Damage (see Sections 2.2.4.1 and 4.3).

Damaging earthquake. Earthquake event that led to the building's assessment for repair needs.

Damaging earthquake shaking. Estimated shaking intensity or ground motion at or near the building site in the damaging earthquake event.

Disproportionate earthquake damage. Damage meeting the criteria defined in the IEBC that indicates excessive damage, given the level of shaking; used in determining the Building Repair Outcome (see Section 4.4.2).

Detailed inspection. Visual inspection at identified inspection locations that exposes the surface of structural components and may involve destructive removal of nonstructural finishes (see Sections 2.2.3.2 and 3.6).

E

F

Fatigue Damage Category. Classification of reinforced concrete components based on damage and reinforcement provided, indicating the likely presence of fatigue damage or the need for further assessment (see Section 5.6.1).

G

Gravity-essential component. A component with damage that could have high consequence in terms of loss of vertical-load-carrying capacity. The definition of such components depends on the structural material and structural system and is found in the material chapters (see Sections 2.2.4.2 and 4.3).

H

I

Initiation of component strength loss. Point on cyclic envelope corresponding to peak strength (see Figure 2-7).

Inspection location. Location identified for detailed inspection based on preliminary inspection, analysis of demands exceeding Inspection Indicators, or locations always requiring inspection (see Section 3.5).

Inspection Indicator. Component force or deformation limit that, if exceeded, indicates that component as an inspection location (see Sections 2.2.3.2 and 3.5.5).

Intrusive inspection. An inspection that may involve removal of structural materials to expose a structural component not otherwise visible and nondestructive testing or other methods depending on the structural material, structural element, and the expected loads
(see Sections 2.2.3.3 and 3.8).

J

K

L

M

N

O

P

Performance-Critical Damage. Damage that leads to a reduction in component and building strength or deformation capacity. As a result of such damage, the future seismic performance of the building is impaired, implying elevated collapse risk and amplified drift demands, relative to the pre-earthquake condition, and indicating the need for performance-critical repairs (see Section 2.2.2).

Performance-Critical Limits. Component force or deformation associated with median (50%) likelihood that DS2 is exceeded (see Section 2.2.5.2).

Performance-critical repairs. Repairs required to address Performance-Critical Damage, i.e. repairs undertaken to restore structural components to their pre-earthquake condition in terms of strength and deformation capacity (see Section 2.2.6.1).

Performance objective. A statement about a target performance level (or levels) of the structure that determines the post-earthquake repair and retrofit needed; the primary performance objective targeted in this document is to restore a structure to its pre-earthquake condition in terms of strength and deformation capacity.

Preliminary inspection. An inspection involving a site visit with visual inspection of the exterior and interior of the building, and that could include nondestructive removal of nonstructural finishes (see Sections 2.2.3.1 and 3.3).

Q

R

Repair. Reconstruction of a structural component or element or of a structure that corrects damage without substantial increases in stiffness, strength, and/or deformation capacity of a load path, or changes to a load path.

Retrofit. Changes to a structural component or element or to a structure that increases stiffness, strength, and/or deformation capacity of a load path, or changes a load path.

S

Substantial structural damage. Damage meeting the criteria defined in the IEBC as modified in these *Guidelines*; used in determining the Building Repair Outcome (see Section 4.4.3).

Sufficiently exposed bar. A reinforcing bar that is exposed by cover spalling around 50% or more of its circumference for a continuous length of four times the bar diameter ($4d_b$) or more. Used for the purpose of determining if bar bucking has occurred during the damaging earthquake (see Section 5.6.1).

T

U

V

Visual Damage State (VDS). Damage State determined through comparison of observed damage with photos from experimental tests from similar components in the electronic Visual Damage States databases (see Section 2.2.5.1).

W

X

Y

Z

Notation

The following definitions are provided to explain notation used in these *Guidelines*. The definitions are organized by uppercase notations, lowercase notations, and Greek notations.

A_{cv} = gross area of concrete section bounded by web thickness and length of section in the direction of shear force considered in the case of walls

A_g = *cross-sectional area of member*

$A_{sh,provided}$ = *area of confinement reinforcement provided in boundary elements of walls*

$A_{sh,required}$ = *area of confinement reinforcement required by ACI 318 for structural walls with special boundary elements*

A_{sl} = *spliced longitudinal reinforcement area*

A_{vsp} = *area of transverse reinforcement provided along the splice length*

B = *failure mode in frame sub-assembly, failure occurs by flexural hinging in the beam without joint failure*

BJ = *failure mode in frame sub-assembly, failure occurs in the joint after flexural hinging in the beam*

C = *failure mode in frame sub-assembly, failure occurs by flexural hinging in the column without joint failure*

C_i = *inspection factor specified in Table 3-1*

CJ = *failure mode in frame sub-assembly, failure occurs in the joint after flexural hinging in the column*

$D_{5\text{-}95}$ = *5% to 95% significant duration*

DC = *component Damage Class*

DC2 = *component Damage Class 2, indicating Performance-Critical Damage*

DCR = *demand-to-capacity ratio*

DS = *component Damage State*

IL = *inspection location*

L_c = *shear span, i.e., the distance of the critical section from the point of contraflexure (also equivalent to a in Appendix B and the VDS databases)*

L_{sp} = *strain penetration length =* $0.15 f_{ye} d_b$ *where the units of* f_{ye} *are ksi, or* $0.022 f_{ye} d_b$ *if the units of* f_{ye} *are MPa*

M = *parameter used in calculating viscous damping ratio for earthquake-damaged systems*

$M/(Vl_w)$ = *ratio of base moment-to-base shear normalized by the wall length*

MCE_R = *Risk-targeted Maximum Considered Earthquake*

M_{ne} = *flexural capacity of coupling beam using expected material properties*

P = *expected compressive axial load for columns and walls*

P_{asb} = *boundary element reinforcement ratio*

S_{a1} = *best estimate of spectral acceleration value at 1 s during the damaging earthquake*

$S_{a1,MCER}$ = *spectral acceleration value at 1 s associated with the* MCE_R *ground motion*

$T_{initial}$ = *first-mode period calculated for the building using effective stiffness following ASCE/SEI 41*

$T_{modified}$ = *first-mode period calculated for the building with component stiffness modified by* λ_k

T_{s1}, T_{s2} = *tension force developed in longitudinal reinforcement of beams, used in classification of frame subassembly and determination of horizontal joint share (Figure 5-6)*

V_{col} = *shear demand in column, used in classification of frame subassembly and determination of horizontal joint share (Figure 5-6)*

V_g = *best-estimate of gravity shear demand in slab-column connections*

V_o = *punching shear strength of slab-column connections per ACI 318 using expected material properties*

V_{ne} = *expected shear strength per ACI 318 for walls and coupling beams using expected material properties*

V_{nfe} = expected *shear-friction* strength of walls *per ACI 318, using expected material properties*

V_{nj} = *expected joint shear strength per ASCE/SEI 41, using expected material properties*

V_p = *beam-column shear demand associated with expected flexural strength*

V_s = *shear strength provided by shear reinforcement*

V_{test} = *shear strength measured in test, applies to beam-column joints*

V_{uj} = *joint horizontal shear demand*

$V_{@Mne}$ = *expected wall shear demand associated with expected flexural strength*

$V_{@Mpb}$ = *expected wall shear demand associated with yielding of beam longitudinal reinforcement using* $f_s = 1.25f_{ye}$

V_0 = *expected undegraded shear capacity of beam-columns*

Z = *reliability index associated with reliability assessment of component deformation demand and deformation capacity;* $Z = 1.28$ *for definition of Inspection Indicator*

a = *ASCE/SEI 41 modeling parameter a; parameter used to measure plastic deformation capacity in component load–deformation curves*

b = *width of compression zone*

c = *neutral axis depth, calculated in accordance with ACI 318 using expected material properties*

c_b = *the lesser of: (1) the distance from center of a bar or wire to nearest concrete surface, and (2) one-half the center-to-center spacing of bars or wires being developed*

d = *ASCE/SEI 41 modeling parameter d; parameter used to measure total (elastic + plastic) deformation capacity in component load–deformation curves*

d = *effective depth of cross-section, distance from extreme compression fiber to centroid of tension reinforcement*

d_b = *longitudinal bar diameter*

f'_{ce} = *expected compressive strength of concrete*

f_{ue} = *expected ultimate tensile strength of reinforcement*

f_{ye} = *expected yield strength of reinforcement*

f_{yel} = *expected yield strength of longitudinal reinforcement*

f_{yet} = *expected yield strength of transverse reinforcement*

h = *total depth of the beam-column cross-section, sometimes distinguished as h_b for beam height and h_c for column height*

k_1, k_2 = *parameters used to determine the approximate depth to the neutral axis for walls*

k_{lp} = *parameter used in component-specific screening check for beams*

l_d = *development length per ACI 318 using expected material properties*

l_n = *coupling beam clear span*

l_{splice} = *provided length of lap splice*

l_w = *length of wall*

m = *component capacity modification factor (m-factor) from ASCE/SEI 41 and modified, as applicable, in Chapter 5 for the primary system Collapse Prevention (CP) limit state*

p = *probability the demand on component exceeds its capacity, i.e. probability that θ_D exceeds θ_{CSL}; $p = 0.10$ for definition of Inspection Indicator*

s = *center-to -center spacing of longitudinal or transverse reinforcement*

s_{joint} = *spacing of hoops in a beam-column joint*

β_{CSL} = *uncertainty in estimation of component deformation capacity at initiation of component strength loss (quantified with logarithmic standard deviation)*

β_{gm} = *uncertainty associated with ground motion record-to-record variability (quantified with logarithmic standard deviation)*

β_{model} = *uncertainty associated with structural analyses and model demand estimation (quantified with logarithmic standard deviation)*

β_{total} = *total uncertainty, considering uncertainty in estimation of demand and capacity*

ε_y = *probable yield strain of longitudinal reinforcement*

Φ = *standard normal cumulative probability distribution*

η = *a multiplier specified in Section 5.4 to adjust between ASCE/SEI 41 modeling parameters (a or d) and estimate of deformation at DS2*

I_p = *Inspection Indicator, defined in terms of plastic rotation or deformation where modeling parameter a is specified in ASCE/SEI 41*

I_t = *Inspection Indicator, defined in terms of total rotation or deformation where modeling parameter d is specified in ASCE/SEI 41*

λ_k = *stiffness modifier applied to the effective stiffness from ASCE/SEI 41*

μ = *estimate of the chord rotation ductility demand from the prior earthquake calculated as the ratio of the peak chord rotation demand to the yield chord rotation*

$\tilde{\theta}_D$ = *median deformation demand on a component*

$\tilde{\theta}_{CSL}$ = *median deformation at point of component strength loss*

ρ = *tension longitudinal reinforcement ratio in beams*

ρ' = *compression longitudinal reinforcement ratio in beams*

ρ_l = *total longitudinal reinforcement ratio for columns*

ρ_t = *transverse reinforcement ratio for beam-columns*

ρ_{wh} = *horizontal web reinforcement ratio in walls*

ζ = *viscous damping ratio recommended for earthquake-damaged systems*

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