Earthquake Damage Assessment and Repair Guidelines for Residential Wood-Frame Buildings

Volume 2 | ENGINEERING GUIDELINES

CEA CALIFORNIA EARTHQUAKE AUTHORITY™

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Earthquake Damage Assessment and Repair Guidelines for Residential Wood-Frame Buildings, Volume 2 – Engineering Guidelines

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Notice

Cover photographs: Victorian house damaged in the 2014 South Napa earthquake (photo credit: CEA).

Preface

In 2018, the California Earthquake Authority (CEA) funded a project with the Applied Technology Council (ATC) to develop guidelines for the assessment and repair of residential wood-frame buildings damaged by earthquakes. This building class represents the most common type of dwelling in California and other regions of the United States. Although wood-frame construction has generally provided good performance in past earthquakes, wide-spread damage to these types of buildings is nonetheless expected in large, future earthquakes centered near urban areas. Additionally, past earthquakes in California have demonstrated the need for more efficiency, consistency, and reliability in the earthquake damage assessment process for residential wood-frame buildings by engineers, insurance claim adjusters, and contractors.

This ATC-143 Project built upon previous work funded by the CEA and others and conducted through the Consortium of Universities for Research in Earthquake Engineering (CUREE) that led to the publication of two documents for the assessment and repair of earthquake-damaged residential wood-frame buildings. The first document, CUREE EDA-06, *Engineering Guidelines for the Assessment and Repair of Earthquake Damage in Residential Woodframe Buildings*, *Version 2005-4* (CUREE, 2005), was published in 2005, is geared towards geotechnical engineers, and includes guidance for making repair and mitigation recommendations for earthquake-induced permanent ground deformation. The second document, CUREE EDA-02, *General Guidelines for the Assessment and Repair of Earthquake Damage in Residential Woodframe Buildings* (CUREE, 2010), was originally published in 2007, was updated in 2010, and is geared towards insurance claim adjusters, contractors, and homeowners. ATC is indebted to all of the individuals involved in this CUREE work, in particular Dan Dyce, John Osteraas, Robert Reitherman, Jonathan Stewart, and Jon Wren, whose leadership and authorship were essential for the development of the original documents.

The ATC-143 Project updated and expanded both documents. In particular, the project expanded the *Engineering Guidelines* by developing guidance for repair of structurally significant earthquake damage to structural elements and to structural bracing of certain nonstructural components, making the document multi-disciplinary in scope and relevant to both structural and geotechnical engineers. As part of this effort, the original geotechnical material was reviewed, reorganized, and in some cases expanded to include recent advances in the practice and to make its presentation consistent with the format and approach of the updated document. The project also took a fresh look at the *General Guidelines* to incorporate feedback from users and lessons learned from recent earthquakes and to ensure alignment with and the updated *Engineering Guidelines*. The *General Guidelines* repair tables were expanded to cover additional building elements and were updated to be consistent with damage thresholds presented in the *Engineering Guidelines.* The design of the *General Guidelines* checklists was revised to make them more user friendly and streamlined, and their content was updated for consistency with changes to other sections of the document. The updated documents are now more integrated and are presented as a two-volume series, *Earthquake Damage Assessment and Repair Guidelines for Residential Wood-Frame Buildings*. The updated *General Guidelines* are published as CEA-EDA-01, *Earthquake Damage Assessment and Repair Guidelines for Residential Wood-Frame Buildings*, *Volume 1 – General Guidelines* (CEA, 2020).

ATC is indebted to the Project Technical Director Morgan Griffith and the Associate Technical Director John Osteraas for their leadership and technical expertise, and to the members of the Project Technical Committee, consisting of David Bonowitz, Kelly Cobeen, David Cocke, and Dan Dyce, for their authorship, review, and technical expertise. ATC would like to thank the members of the Project Review Panel, consisting of Robert Reitherman (Chair), Lyle Carden (ATC Board Contact), Warner Chang, Tara Hutchinson, Bret Lizundia, Lisa Lohmann, David Ojala, and Frank Rollo, who provided advice, review, and assistance at key stages of the work. ATC also would like to thank Working Group Members Sean Ahdi, Christine Beyzaei, and Jon Wren for their work on the geotechnical material in the *Engineering Guidelines*; Taylor Funk and Kari Klaboe for, among other things, their test runs of draft versions of the structural damage assessment and repair procedures in the *Engineering Guidelines*; and Evelyn Mikailian for her help in reviewing and identifying photos; as well as the more than 30 structural and geotechnical engineers who participated in a review workshop of the *Engineering Guidelines* in the San Francisco Bay Area in September 2019. The names and affiliations of the workshop participants are provided in the list of Project Participants at the end of this report.

ATC also gratefully acknowledges funding provided by the CEA and the guidance and support provided by Janiele Maffei (CEA Chief Mitigation Officer), Mitch Ziemer (CEA Insurance and Claim Director), and Erin Rogers (CEA Project Coordinator). Carol Singer provided graphic design services, Ayse Hortacsu (ATC) provided project management services, and Carrie J. Perna (ATC) provided report production services.

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CHAPTER 1

Introduction

This document is the second of two volumes comprising the series *Earthquake Damage Assessment and Repair Guidelines for Residential Wood-Frame Buildings*. The documents describe the process of identifying, evaluating, and repairing common earthquake damage in typical residential wood-frame buildings and are intended to increase the efficiency, consistency, and reliability of the earthquake damage assessment and repair process. They provide guidance on issues related to:

- common earthquake vulnerabilities and damage potential in houses,
- identification and documentation of earthquake damage,
- evaluation of earthquake damage, and
- development of a conceptual scope of repair for earthquake damage.

The focus of these documents is typical one- and two-family wood-frame dwellings, generally referred to herein as "houses." While manufactured homes (mobile homes) and larger multi-unit residential buildings are outside the scope of the documents, the general principles and methodology may still be applicable for these buildings. Some of the details and references are California-centric, but the general principles and methodology are applicable throughout the United States. It is not intended that use of these documents is limited to any particular building configuration or location; rather, the intent is to inform users that certain building characteristics were contemplated in the organization of the documents and the development of their content.

This document, referred to herein as the *Engineering Guidelines*, is primarily intended for use by technical consultants, such as engineers and architects, who are involved in post-earthquake damage assessment and repair of wood-frame buildings. The companion document to the *Engineering Guidelines* is CEA-EDA-01, *Earthquake Damage Assessment and Repair Guidelines for Residential Wood-Frame Buildings*, *Volume 1 – General Guidelines* (CEA, 2020), referred to herein as the *General Guidelines*. The intended audience of the *General Guidelines* is building contractors, insurance claim representatives (adjusters), homeowners, and others familiar with construction and repair. The *General Guidelines* also serve as a reference for technical consultants engaged in post-earthquake damage assessment and repair of wood-frame buildings.

The *Engineering Guidelines* are intended to provide guidance to the technical consultant retained by a user of the *General Guidelines* to identify and develop a conceptual scope of repair for structurally significant earthquake damage. In some cases, earthquake shaking can result in earthquake-induced permanent ground deformation that may cause damage to buildings or associated surface improvements. The *Engineering Guidelines* are also intended to provide guidance to the technical consultant who has been retained to identify and develop a conceptual scope of repair and mitigation for earthquake-induced permanent ground deformation.

1.1 Intended Audience and Use of the Engineering Guidelines

A technical consultant is part of a larger team in the process of identifying and repairing earthquake damage. Figure 1-1 presents a flowchart that identifies the key steps and their relationships in the post-earthquake damage assessment process envisioned by the *Engineering Guidelines*. The assessment performed under the *General Guidelines*, prior to the retention of the technical consultant, is represented at the top of the flowchart. For the purposes of the *Engineering Guidelines*, it is assumed that the technical consultant has been retained to address conditions identified as potentially structurally significant earthquake damage based on an examination of the building in accord with the *General Guidelines*. In that context, the technical consultant's objective is to determine if the observed damage is structurally significant and, if so, to describe the appropriate conceptual repair. Modification to this scope of work can be appropriate, and, in the case of a more limited scope (e.g., where the consultant has been retained to provide a second opinion regarding a particular condition), only certain sections of the *Engineering Guidelines* may be applicable. In the case of a more expansive scope (e.g., where the consultant has been retained to provide conceptual repair recommendations for both earthquake damage that is structurally significant and earthquake damage that is not), the technical consultant may choose to rely on procedures of the *General Guidelines* in making repair recommendations for damage that is not structurally significant.

The *Engineering Guidelines* address two types of technical consultants: structural and geotechnical. Structural consultants are primarily concerned with the assessment and repair of earthquake damage to the building, while geotechnical consultants are primarily concerned with the assessment and repair or mitigation of earthquake-induced permanent ground deformation affecting the building and associated surface improvements. Structural consultants generally include structural engineers and civil engineers with expertise in structural engineering, as well as architects with related expertise. Geotechnical consultants generally include geotechnical engineers and civil engineers with expertise in geotechnical engineering, as well as engineering geologists with related expertise. One or both types of consultants can be retained initially; however, since the building itself is the primary concern in most cases, it is common practice for a structural consultant to be retained initially and for the structural consultant to recommend the retention of a geotechnical consultant as needed.

Minimum qualifications of the technical consultant include professional licensure in the state where the property is located, familiarity with both the *General Guidelines* and the *Engineering Guidelines*, and an understanding of the unique aspects of residential wood-frame construction, including typical elements along the load path of the seismic-force-resisting system.

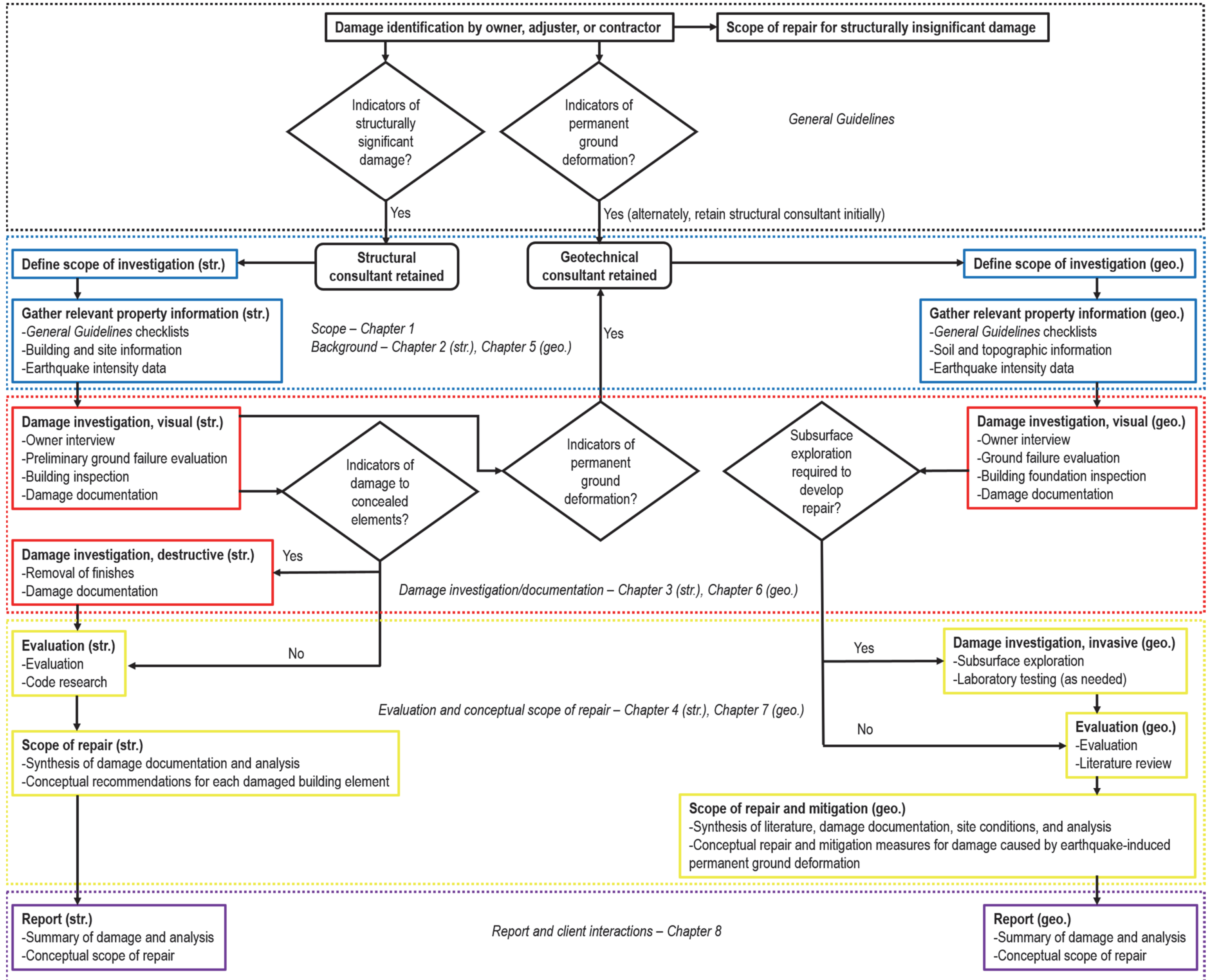


Figure 1-1 Post-earthquake damage assessment process envisioned by the *Engineering Guidelines*.

1.2 Earthquake Damage Assessment and Repair Methodology

In this document, earthquake damage is defined as an adverse, non-trivial, physical change in the safety, serviceability, appearance, or repairability of an element or portion of a house caused by earthquake ground shaking or earthquake-induced permanent ground deformation. Damage is considered structurally significant when it results in a non-trivial, adverse change in the ability of a house or any of its elements to sustain load or resist future earthquake shaking, whether caused by structural or geotechnical damage mechanisms. All other damage to a house is considered structurally insignificant. The *Engineering Guidelines* address the identification and repair of structurally significant earthquake damage with the presumption that structurally insignificant earthquake damage will be addressed by users of the *General Guidelines*.

The basic objective of the repairs recommended in the *Engineering Guidelines* is to restore a house to its pre-earthquake condition using established engineering principles and methods of construction. The *Engineering Guidelines* recommend repairs for structurally significant earthquake damage to structural elements and to the structural bracing of certain nonstructural components. With respect to a building's gravity-load-resisting and seismic-force-resisting systems, the *Engineering Guidelines* recommend repairs intended to provide structural performance that is substantially equivalent to the pre-earthquake condition in terms of the expected damage states and related safety during future earthquakes. Other considerations, such as application of particular insurance policy terms or decisions regarding building replacement or alteration instead of repair, are outside the scope of this document. This document specifically does not address repair of damage or deficiencies resulting from non-earthquake causes; discussions regarding such conditions are included in the *General Guidelines*.

Consistency with Building Codes

Repair of earthquake damage to houses is typically regulated by a locally adopted building code. Throughout the United States, the applicable code is an adopted version of a recent edition of either the *International Existing Building Code* (IEBC) (ICC, 2018a) or the *International Residential Code* (IRC) (ICC, 2018b). The repair objective expressed in the *Engineering Guidelines* is generally consistent with the intent of both the IEBC and the IRC as they are currently adopted in California. In some cases, the local code triggers additional work beyond what the *Engineering Guidelines* recommend. These cases include triggered upgrades (discussed in Appendix A) and local interpretations of model code provisions, which are outside the scope of this document.

Where earthquake-induced permanent ground deformation is determined to have caused damage to a house or associated surface improvements, the *Engineering Guidelines* provide guidance on the evaluation and identification of repair and mitigation strategies using established engineering principles. The objective of the repair and mitigation strategies recommended in the *Engineering Guidelines* is to address the earthquake-induced permanent ground deformation, so that a house and associated surface improvements can be restored to their pre-earthquake conditions. The *Engineering Guidelines* recommend repair and mitigation measures for earthquake-induced permanent ground deformation that are appropriate for the magnitude of ground deformation and collateral physical damage observed at the property, and that account for the potential geotechnical earthquake hazards that may still exist at the property.

In some instances, it may be appropriate for a technical consultant to develop a scope of repair that incorporates upgrades relative to the pre-earthquake condition of a house because upgrades are required by the authority having jurisdiction (i.e., code-triggered upgrades) or because upgrades are desirable for improving future seismic performance (i.e., voluntary upgrades). These cases are discussed in Appendices A and B, respectively.

The focus of the *Engineering Guidelines* is earthquake-related *damage assessment*, not *safety assessment*. As discussed in Section 1.2 of the *General Guidelines*, a safety assessment for the purposes of building occupancy may have occurred prior to the technical consultant's work, resulting in an unsafe (red), restricted use (yellow), or inspected (green) "tag," or placard, posted on a house. Although not addressed in detail by the *Engineering Guidelines*, emergency (temporary) repairs to a house can allow for continued occupancy of the building pending final repairs, particularly in the case of red and yellow postings. During a damage assessment, the technical consultant may identify unsafe conditions that could be temporarily addressed by emergency repairs, and it may be appropriate to recommend such repairs to the client.

1.3 Organization and Content

This document consists of eight chapters and a series of appendices:

Chapter 1 describes the scope and intended use of the document.

Chapter 2 provides an overview of earthquake effects on houses. This chapter is intended primarily for the structural consultant.

Chapter 3 provides guidance for on-site identification and documentation of earthquake-induced building damage. This chapter is intended primarily for the structural consultant.

Chapter 4 provides guidance for the evaluation and repair of earthquake-induced building damage. This chapter is intended primarily for the structural consultant.

Chapter 5 provides an overview of earthquake-induced permanent ground deformation mechanisms. This chapter is intended primarily for the geotechnical consultant.

Chapter 6 provides guidance for on-site identification and documentation of earthquake-induced permanent ground deformation. This chapter is intended primarily for the geotechnical consultant.

Chapter 7 provides guidance for the evaluation and repair and mitigation of earthquake-induced ground deformation. This chapter is intended primarily for the geotechnical consultant.

Chapter 8 provides guidance for determining an appropriate scope and format for technical consultant reports.

Appendix A through Appendix D are intended primarily for the structural consultant. Appendix A provides a brief introduction to code-triggered upgrades. Appendix B includes an annotated bibliography of available information to assist structural consultants in voluntary seismic upgrades. Appendix C summarizes information from laboratory testing about earthquake damage to concealed elements. Appendix D presents available research illustrating relationships between earthquake damage patterns and the strength and stiffness of elements of the seismic-force-resisting system.

Appendix E through Appendix I are intended primarily for the geotechnical consultant. Appendix E contains a short description of analysis for evaluating if observed ground deformations are related to distributed surface rupture. Appendix F provides descriptions of liquefaction screening, triggering and consequences analyses. Appendix G summarizes procedures for estimating ground displacements from seismic compression. Appendix H contains a description of analyses for seismic slope stability. Appendix I includes information about analyses related to the deformation and stability of retaining walls.

Appendix J includes a compilation of the investigation checklists presented in Chapter 3. An appendix with a Glossary and Acronyms—providing definitions of key terms used in this document—and a list of References are provided at the end of the report.

CHAPTER 2

Background Information: Structural

2.1 Introduction

This chapter provides background material relevant to the assessment of earthquake damage to a house. It is intended to complement more procedural material in Chapter 3 and Chapter 4. Section 2.2 describes earthquake vulnerabilities commonly encountered in houses and their associated earthquake damage mechanisms, which in turn can affect many structural elements of houses. Section 2.3 describes common construction practices for various structural elements of houses, as well as typical earthquake damage patterns specific to those elements. For ease of reference, Section 2.3 is arranged in subsections that match the subsections of Chapter 3 and Chapter 4. Finally, Section 2.4 is a discussion of earthquake intensity and its relationship to damage potential.

2.2 Potential Earthquake Vulnerabilities in Houses

Traditional house construction has characteristics that make some houses vulnerable to certain earthquake damage mechanisms, such as the sliding of walls relative to their foundations. This section presents commonly encountered earthquake damage mechanisms, organized by configuration vulnerabilities in houses that enable the mechanisms to occur. An understanding of these mechanisms in relation to a house's overall configuration is expected to help a user of the *Engineering Guidelines* find damage, evaluate it, and recommend appropriate repairs following an earthquake.

2.2.1 Inadequate Anchorage of Superstructures at Foundations

The sill plates of older houses were not always (or adequately) attached to the concrete or masonry foundation (see Section 2.3.4). Even in more modern houses, sill plates of interior walls may have minimal attachment to a foundation or floor slab. During earthquakes, these sill plates can slide relative to the foundation or floor slab, resulting in residual displacement, splitting of the sill plates, damage to the wall finishes or framing, and misalignment of doors and windows along displaced walls. As discussed in some of the references cited in Appendix B, adding steel post-installed anchor bolts along unanchored sill plates is a common retrofit measure to mitigate this earthquake vulnerability.

Figure 2-1 shows substantial sliding, readily observed, with fracture of the sill plate and conspicuous damage to the wall finishes along the sill plate. Figure 2-2 shows less obvious sliding, initially noticed only because of inoperable doors within the affected wall line.

Figure 2-1 Earthquake-caused sliding of superstructure at foundation. The wall has shifted to the right several inches relative to the foundation, stucco wall finishes have cracked and spalled along the sill plate, and the sill plate is fractured near the right end of the wall (photo credit: Exponent).

Figure 2-2 Earthquake-caused sliding of superstructure at floor slab. The wall has shifted less than 1 inch to the left, as evidenced in the left photo by the tapered gap between the left side of the door and doorjamb and in the right photo by the gap between the carpet and doorjamb (photo credit: Exponent).

2.2.2 Unbraced Cripple Walls

Where a cripple wall is inadequately braced, permanent drift over the height of the cripple wall is a common (and costly) earthquake damage mechanism. Where the drift is moderate, it is typically associated with damage to the wall finishes that provide nominal strength and stiffness to an otherwise unbraced cripple wall (Figure 2-3), damage to the cripple wall framing, and damage to connections between the cripple wall and the wall or floor framing above. If the in-plane cripple wall drift in one direction becomes large, the cripple walls in the other direction will lean out of plane and be prone to collapse (Figure 2-4). In addition to structural damage, large cripple wall drift or collapse can damage mechanical, electrical, and plumbing (MEP) components commonly located in the crawlspace. As discussed in references cited in Appendix B, adding plywood or oriented strand board (OSB) along the interior faces of cripple walls is a common retrofit measure to improve the earthquake performance of houses with respect to this earthquake vulnerability.

Figure 2-3 Damage to cripple wall finishes caused by the 2014 South Napa, California earthquake (photo credit: Exponent).

Figure 2-4 Cripple wall collapse caused by the 1994 Northridge, California earthquake resulting in widespread related damage in upper-story framing and relative movement between the main house and the covered porch (photo credit: Federal Emergency Management Agency, or FEMA).

2.2.3 Hillside Houses

Hillside houses typically incorporate stepped or sloped perimeter foundations (or stemwalls) along their sides (Figure 2-5). Posts or wood-frame walls enclosing the underfloor area along these lines may range in height from twenty or more feet at the downhill end to effectively zero at the uphill end, where the floor framing is typically attached to the uphill foundation with a sill plate or ledger. The tall posts or walls, if inadequately braced, may allow excessive deformation at the downhill end, causing torsional response and increasing demands on the uphill framing-to-foundation connections. This damage mechanism typically includes framing deformation and damage (Figure 2-5), and in the worst cases, localized loss of support for floor framing at the uphill end (Figure 2-6) or complete separation of the house from the uphill foundation, resulting in collapse. Hillside houses with no sidewalls, or only short sidewalls at the uphill end, are particularly prone to this damage mechanism. In addition to structural damage, MEP components located below the house may also be damaged.

Figure 2-5 Damaged framing enclosing the underfloor area of a hillside house caused by the 2003 San Simeon, California earthquake. The damage includes detachment of the stucco from the wall framing and both in-plane racking and out-of-plane leaning of the wood-frame walls (photo credit: Exponent).

Figure 2-6 Damage at uphill end of a hillside house caused by the 2003 San Simeon, California earthquake. The floor framing detached and pulled away from the uphill foundation, causing the floor to sag (photo credit: Exponent).

2.2.4 Narrow Wall Piers

Where window and door openings leave only narrow wall segments to resist earthquake forces, wall lines can be prone to excessive in-plane drift (Figure 2-7) and possibly collapse. Where wider wall segments are not possible, newer houses avoid this vulnerability by using portal frame systems, proprietary engineered wall piers, or even structural steel elements. This mechanism can result in damage to wall pier finishes and sheathing, damage to wall pier framing, damage to connections at the tops and bottoms of wall piers, and inoperability of doors and windows within racked wall lines.

Figure 2-7 Large in-plane drift of narrow wall piers at garage door opening caused by the 1971 San Fernando, California earthquake (photo credit: National Information Service for Earthquake Engineering, or NISEE).

2.2.5 Living Spaces Over Garages

Multi-story houses with attached garages often have living spaces directly over relatively open garage areas. Where only narrow wall segments are provided on either side of a wide garage door, the additional weight of the upper story can make an already weak condition even more prone to excessive drift (Figure 2-8) and possibly collapse.

Figure 2-8 Damage related to living space over garage caused by the 2014 South Napa, California earthquake. In-plane racking of the front wall line of the garage caused cracking and possible framing damage in the narrow wall segments on either side of the garage door (photo credit: Exponent).

2.2.6 Framing Discontinuities

Earthquake damage is often concentrated where structural framing is discontinuous from one story to the next or between wings of a house in plan. Earthquake damage associated with these framing discontinuities includes concentrated damage to finishes, residual separations, and framing damage. In split-level houses, where the floor or roof framing is discontinuous between the one-story and two-story portions, differential movement and lack of effective connection between the portions can lead to damage. Where the common wall between the portions provides support for the lower-story floor or roof, the separation can result in local collapse. Where two wings of a house join, relative movement between the wings can lead to damage along the intersection (Figure 2-9).

Figure 2-9 Left: Adjoining wings of a house create a framing discontinuity. Right: Separation of wood siding at intersection of adjoining wings caused by the 2014 South Napa, California earthquake. The separation was not structurally significant because there was no continuous framing or structural connection between the wings (photo credit: D. Bonowitz).

2.3 Potential Earthquake Damage to Structural Elements of Houses

2.3.1 Relationship between Vulnerabilities and Damage to Structural Elements of Houses

This section describes common structural elements of houses and their common earthquake damage patterns. The intent is to provide background information for the structural consultant performing the damage finding and documentation procedure described in Chapter 3. Section 2.3.2 through Section 2.3.10 below align with Section 3.2 through Section 3.10 in Chapter 3. Reference is made to additional background information on structural elements provided in the *General Guidelines*.

Damage to individual elements is frequently the consequence of earthquake damage mechanisms enabled by configuration vulnerabilities. Table 2-1 highlights these relationships. For each configuration vulnerability presented in Section 2.2, Table 2-1 lists the associated structural elements that typically sustain damage during earthquakes.

Table 2-1 Relationship Between Configuration Vulnerabilities and Associated Structural Elements that Typically Sustain Damage During Earthquakes

Configuration Vulnerability	*Associated Structural Elements*
Inadequate anchorage of superstructures at foundations (Section 2.2.1)	Sill plates and anchorage (Section 2.3.4) Wood-frame walls (Section 2.3.5)
Unbraced cripple walls (Section 2.2.2)	Foundations and slabs-on-grade (post and pier) (Section 2.3.3) Wood-frame walls (Section 2.3.5)
Hillside houses (Section 2.2.3)	Sill plates and anchorage (hillside houses: framing-to-foundation connections) (Section 2.3.4) Wood-frame walls (Section 2.3.5) Floors, ceilings, and roofs (floor framing, floor diaphragms) (Section 2.3.7)
Narrow wall piers (Section 2.2.4)	Wood-frame walls (Section 2.3.5) Other seismic-force-resisting elements (Section 2.3.6)
Living spaces over garages (Section 2.2.5)	Wood-frame walls (Section 2.3.5) Other seismic-force-resisting elements (Section 2.3.6)
Framing discontinuities (Section 2.2.6)	Wood-frame walls (Section 2.3.5) Floors, ceilings, and roofs (Section 2.3.7)

In typical houses, earthquake forces are collected in floor and roof diaphragms and transferred to the foundations by sheathed walls. Figure 2-10 illustrates the framing and sheathing elements in a typical house.

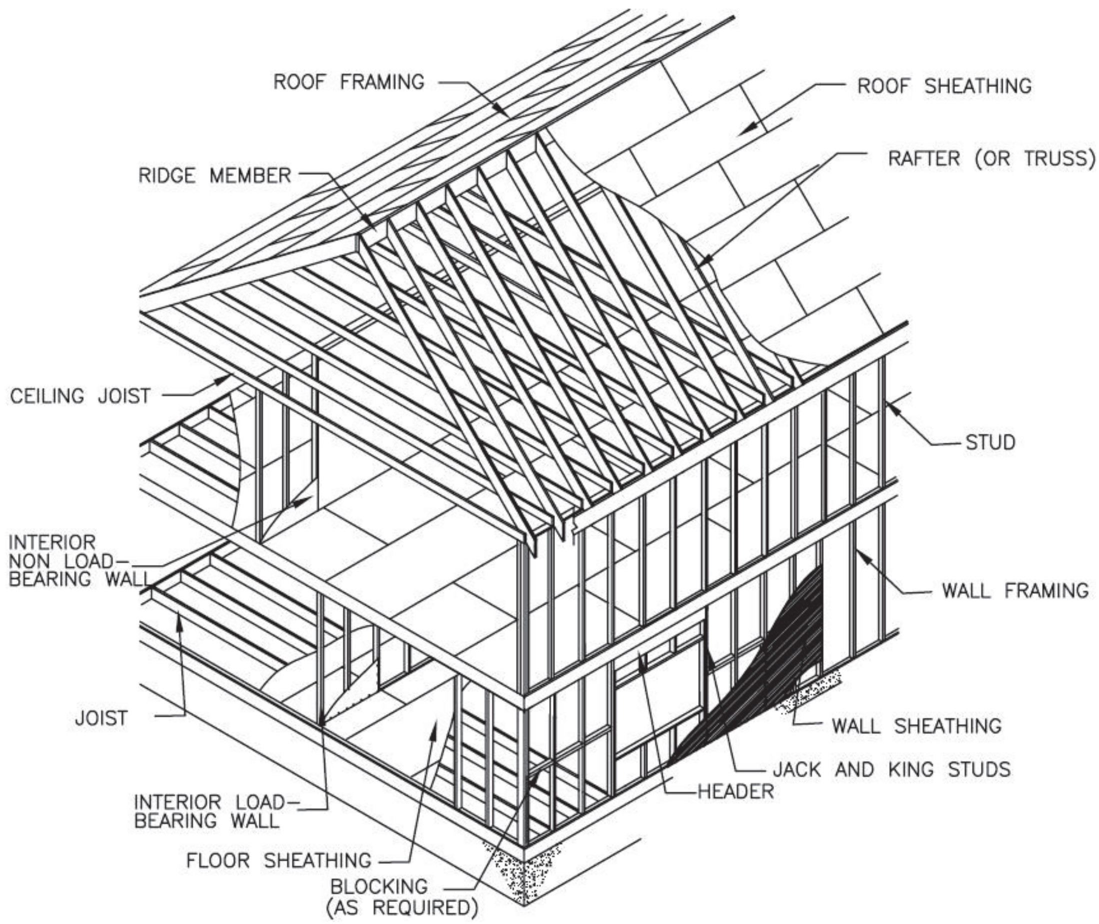


Figure 2-10 Structural elements of a typical house (image credit: U.S. Department of Housing and Urban Development).

2.3.2 Ground

During earthquakes, houses and other site improvements can be damaged directly by strong shaking or indirectly by earthquake-induced permanent ground deformation. Additional information related to earthquake and non-earthquake ground deformation mechanisms is presented in Chapter 5. While the information in Chapter 5 is intended primarily for geotechnical consultants, structural consultants should be familiar with evidence of ground deformation mechanisms and their effects on houses and site improvements.

2.3.3 Foundations, Slabs-on-Grade, and Basement Walls

Slabs-on-Grade

In warm locales, concrete slab-on-grade floors are commonly used in houses, typically combined with thickened portions of the slab to provide foundations for load bearing walls. Concrete slabs-on-grade can be reinforced or unreinforced. If the concrete slab is reinforced at all, the reinforcing is typically intended to control shrinkage cracking. Some houses built after about 1970 have post-tensioned slabs to control settlement or heaving from expansive soil. Even so, virtually all slab-on-grade floors, whether exposed in a garage area or covered by carpet or other finishes, contain cracks due to concrete shrinkage and non-earthquake-related differential ground movement.

Earthquake damage to concrete slab-on-grade floors is almost always caused by permanent ground deformation due to one or more of the mechanisms discussed in Chapter 5. This damage typically takes the form of fresh cracks with offsets (vertical or horizontal) or sloping across cracks and joints. These patterns of earthquake damage will also reveal themselves as damage to brittle floor finishes adhered to the slab.

In rare instances, the slab-on-grade moves slightly relative to the concrete footing along its edge if the two are not integrally cast. Although this damage mechanism usually does not result in structurally significant damage, it can result in damage to finishes near the joint between the slab and the footing.

For additional information related to slabs-on-grade, see Section 4.4 through Section 4.6 of the *General Guidelines*.

Footings and Stemwalls, Concrete

Continuous concrete strip footings are commonly used in conjunction with concrete slab-on-grade floors and to support concrete and masonry stemwalls. Footings can be cast integrally with slabs or stemwalls, or footings can be cast first and the slabs or stemwalls cast on top of the footings. In houses with crawlspaces, concrete stemwalls are commonly used to support the floor framing around the perimeters of houses and may be used at interior locations to reduce the spans of floor framing. Concrete stemwalls typically contain nominal steel reinforcing, but concrete footings may not be reinforced in older construction.

Earthquake damage to concrete footings and stemwalls is almost always caused by permanent ground deformation due to one or more of the mechanisms discussed in Chapter 5. This damage typically takes the form of fresh-appearing cracks with offsets (vertical or horizontal) (see Figure 2-11) or sloping across cracks and joints. In cases of significant ground deformation, a concrete stemwall may tilt or slope.

For additional information related to footings and stemwalls, see Section 4.4 through Section 4.6 of the *General Guidelines*.

Figure 2-11 Cracking of concrete stemwall in-line with ground crack resulting from the 2003 San Simeon, California earthquake (left). The offset across the crack is revealed in the photograph on the right (photo credit: Exponent).

Footings and Stemwalls, Masonry

Continuous brick strip footings or brick stemwalls were used in older houses, and these foundations are almost never reinforced. Stemwalls constructed of concrete masonry units and supported by concrete footings are used in some newer houses and may or may not have grouted cells or steel reinforcing. Earthquake damage patterns for these foundation elements are similar to those of concrete footings and stemwalls, except that offsets between masonry units can occur, particularly in the case of unreinforced foundations (see Figure 2-12).

For additional information related to footings and stemwalls, see Section 4.4 through Section 4.6 of the *General Guidelines*.

Figure 2-12 Damage of unreinforced, ungrouted masonry stemwalls caused by the 2011 Virginia earthquake (photo credit: M. Mahoney).

Post and Pier Foundations

Post and pier foundations are commonly used to support the interior portions of floor framing in houses with crawlspaces, and they are typically used in conjunction with perimeter stemwalls or cripple walls. In some older houses, post and pier foundations are also located around the perimeters of buildings in lieu of perimeter stemwalls or framed cripple walls. The most common configuration is a wood post supported on a concrete pier with depth below ground ranging from a few inches to several feet. The wood post may be toe-nailed to the supported floor beam, may be attached to the beam with framing hardware, or may not be attached to the floor beam at all, since it is common to encounter wood shims installed between the post and beam. The post is frequently unattached to the concrete pier, although piers in newer houses may have framing hardware embedded in the top for attachment of the post.

Earthquake damage to concrete piers is almost always caused by permanent ground deformation due to one or more of the mechanisms discussed in Chapter 5. This damage can take the form of cracking or rotation of piers due to movement of the ground. Unattached wood posts can move relative to the floor beams or piers (Figure 2-13) or become dislodged and fall as a result of earthquakes. If posts are attached to both the floor beams and piers, the wood posts or connections can become damaged (Figure 2-14).

For additional information related to post and pier foundations, see Section 4.4 through Section 4.6 of the *General Guidelines*.

Figure 2-13 Shifting of an unattached wood post along the top of the concrete pier due to the 2006 Kiholo Bay, Hawaii earthquake. There was no continuous perimeter foundation around the building (photo credit: Exponent).

Figure 2-14 Leaning posts within the interior of the house plan. The house had a perimeter foundation with inadequate anchorage between the superstructure and the foundation (photo credit: R. Gallagher).

Basement Walls, Concrete

Full basements are rare in warm locales where the frost line depth is only a few inches from the ground surface. In these cases, the most common basements are effectively below-grade closets just large enough to house a boiler or water heater (the so-called California basement) or are built into a sloping site so that the basement walls also retain several feet of soil at the uphill end. In the latter case, the basement "room" is actually a below-grade garage accessed by a sloped driveway. In most cases, concrete basement walls have minimal reinforcing steel and no construction joints, so they are prone to cracking (Figure 2-15).

Basement walls often serve the same purpose as stemwalls and can therefore sustain the same patterns of earthquake damage. Because they also retain soil, basement walls can experience earthquake damage in patterns similar to those discussed in Chapter 5.

Figure 2-15 Typical non-earthquake-related cracking of concrete California basement wall (photo credit: Exponent).

Basement Walls, Masonry

See the discussion of concrete basement walls above. Earthquake damage to masonry basement walls can be similar to that associated with masonry stemwalls (discussed above) or retaining walls (see Chapter 5). Typical brick masonry and concrete masonry unit (CMU) basement wall configurations are depicted in Figure 2-16.

Figure 2-16 Typical brick masonry (left) and CMU (right) basement wall configurations, where cracking in the right photo was caused by the 2018 Anchorage, Alaska earthquake (photo credits: D. Bonowitz, left, and Z. Stutts, right).

2.3.4 Sill Plates and Anchorage

Wood Sill Plates

Wood-frame houses with continuous concrete or masonry foundations have wood sill plates (sometimes referred to as foundation sill plates or mudsills) that bear on the foundations and provide bases for wood wall studs or floor joists (see Figure 2-17). Sill plates in newer houses, particularly along perimeter wall lines, are almost always attached to the foundations with steel anchor bolts embedded into the concrete or masonry. Sill plates in older houses may not have steel anchors but may have been set into the wet concrete to provide some nominal attachment. In urban areas of California, it is rare to find a sill plate lying directly on a concrete footing with no attachment at all.

Earthquake damage to sill plates can include fracture or crushing of the plates associated with rocking of narrow wall piers. Sill plates that are not attached to foundations can experience sliding, as discussed in Section 2.2.1, and in some instances, sill plates bolted to foundations can experience crushing or splitting around anchor bolts.

Figure 2-17 Sill plate below cripple wall framing of older house with steel post-installed anchor bolts attaching the sill plate to the foundation (photo credit: D. Bonowitz).

Steel Cast-in-Place Anchor Bolts

As discussed above, wood sill plates in all but the oldest houses are typically attached to foundations with embedded, or cast-in-place, steel anchor bolts (see Figure 2-18). Earthquake damage associated with steel cast-in-place anchor bolts can include splitting of the wood sill plates, deformation or fracture of the steel anchors, or damage to the concrete or grout surrounding the anchors (localized crushing of a sill plate around a washer can be caused by over-tightening of a nut or an under-sized washer). Fracture of steel anchors is rare and would typically be accompanied by damage patterns associated with sliding of the sill plates, as discussed in Section 2.2.1. Damage to the concrete or grout is typically in the form of fresh cracking or spalling of the concrete or grout around the anchor bolts and is most often associated with anchors installed with inadequate end or edge distances.

Figure 2-18 Typical steel cast-in-place anchor bolt attaching sill plate to concrete foundation (photo credit: E. Mikailian).

Steel Post-Installed Anchor Bolts

Steel anchor bolts may be installed into hardened concrete or grout. Usually, such an installation is part of a seismic retrofit or other alteration made after original construction (Figure 2-19). The bolts can be expansion-type anchors or adhesive anchors. Earthquake damage patterns associated with steel anchor bolts installed into hardened concrete or grout are similar to those for steel cast-in-place anchors. These anchors can be more prone to pull-out failure if not adequately attached to the concrete or masonry.

Figure 2-19 Steel post-installed anchor bolts associated with modifications to a house. These are adhesive anchors attached to an existing concrete foundation (photo credit: E. Mikailian).

Steel Post-Installed Plate Connectors

Steel plate connectors (sometimes referred to as retrofit plates) serve the same purpose as post-installed bolt anchors but are used where installation of bolt anchors is difficult. The steel plates are attached to the sides of sill plates with wood screws and attached to the concrete or masonry stemwalls with steel post-installed bolt anchors (Figure 2-20); the plates typically contain pre-drilled holes for inserting these fasteners into the sill plates and foundations. Earthquake damage associated with steel post-installed plate connectors is most likely to include splitting of the wood sill plates, deformation or fracture of the wood screws, or damage to the steel post-installed bolt anchors (see discussion above). Although unlikely, damage can also include deformation or fracture of the steel plates.

Figure 2-20 Steel post-installed plate connectors attaching a wood sill plate to a concrete stemwall. The connectors were installed as part of a seismic retrofit (photo credit: D. Bonowitz).

Hillside Houses: Framing-to-Foundation Connections

The framing-to-foundation connections at the uphill end of a hillside house (Figure 2-21) can be prone to earthquake damage. These connection details can vary significantly, so earthquake damage can involve wood ledgers, wood sill plates, steel anchors, or other connection assemblies. If the connection is severed completely, the damage can manifest as loss of bearing for the floor joists, with associated damage to floor finishes and adjacent wall finishes.

Figure 2-21 View of the framing connections to the uphill foundation (at right side of the photograph) at a hillside house (photo credit: Exponent).

2.3.5 Wood-Frame Walls

Horizontal Wood Siding

Cyclic testing of horizontal wood siding indicates low strength and stiffness, but with much of the peak strength retained at large drift levels (see Appendix C). This contrasts with the behavior of brittle wall finishes (e.g., stucco, plaster, or gypsum wallboard), which are initially stiff but lose strength at higher drift levels. As a result, when combined with other wall sheathing materials, horizontal wood siding (Figure 2-22) contributes little to wall performance at low-to-moderate drift levels but contributes more significantly at large drift levels. In addition, structurally significant damage to horizontal wood siding will almost always be preceded by significant damage to any brittle materials present in the same wall assembly.

Earthquake damage associated with horizontal wood siding, in order of increasing severity, includes localized splitting of the siding around nails, withdrawal of the nail heads from the siding, detachment of the siding from the wall framing, localized crushing of siding, elongation of nail holes at the back faces of the siding boards, and occasionally splitting of the wall framing at the nails. Walls braced by horizontal wood siding alone (such as at garages and cripple walls where the opposite faces are unfinished) are prone to large residual drift.

Figure 2-22 Horizontal wood siding (shiplap) on the side wall and stucco on the front face of a house with a living-space-over-garage condition (photo credit: D. Bonowitz).

Plywood Panel Siding

Plywood is commonly used as an exterior wall finish material in houses, typically in the form of panels with decorative vertical grooves. Although these panels may not be considered structural sheathing, their earthquake damage patterns are similar to those of wood structural panels described below (Figure 2-23). Plywood panel siding that is edge nailed at all edges of each panel can effectively function as a shear wall. It is more common, however, to find edge nailing along only one of the two vertical panel edges. When installed in this manner, plywood panel siding retains some bracing capacity but much less than if the edges were fully nailed, and additional panel and framing deformation, as well as more frequent nail tear-out at panel edges, can occur with this installation.

Figure 2-23 Plywood panel siding detached from cripple wall framing at a hillside house (photo credit: R. Gallagher).

Stucco

For a house without wood structural panels, an exterior finish of Portland cement plaster, or stucco, often provides the majority of the structure's lateral strength and stiffness. Although stucco can provide considerable bracing to a house, stucco is brittle and therefore prone to cracking. In addition, good performance relies on an adequate connection to the wall framing. Stucco construction involves the fastening of a hex wire lath (similar to chicken wire) through building paper to the wall construction below, and application of the cementitious stucco material in three coats. In construction during the early 1900s, the stucco was often applied over lumber sheathing and fastened using furring nails. Later construction can include application over plywood or OSB sheathing, or fastening directly to the studs, with wires strung across the studs as a backing for plaster application. Later construction also can include use of staples in place of furring nails and, in some cases, use of wire lath pre-attached to building paper (self-furring lath). In all cases, the fastening of the stucco lath is critical to the ability of the stucco to provide bracing for seismic forces.

The most common earthquake damage to stucco finishes includes diagonal cracking emanating from corners of door and window openings (Figure 2-24). As strong shaking continues, the cracks can extend from these openings, transitioning to longer horizontal cracks, and "X" cracks can form across a pier between openings. Spalling of the stucco along cracks can also occur. In more extreme cases, particularly where the lath connection is inadequate, large areas of stucco can detach from the wall framing. Earthquake damage to stucco can also manifest as a horizontal crack at the interface between the sill plate and foundation, associated either with in-plane sliding of the sill plate or out-of-plane deformation of the wall.

Figure 2-24 Stucco cracking and spalling at a door opening due to the 2019 Ridgecrest, California earthquake (photo credit: M. Ziemer).

Another form of earthquake damage to stucco finishes includes detachment of the stucco from the framing, particularly along the bottom edge of the stucco at the top of foundations or stemwalls. Where this occurs, the damage mechanism often extends for a significant distance, possibly the entire length of the wall line. Where detached, the stucco no longer provides effective bracing for earthquake loading.

Stucco also cracks from non-earthquake causes, including shrinkage, often in similar patterns. The corners of door and window openings are particularly prone to shrinkage cracking. Along the sill plate, horizontal cracks can occur due to shrinkage stresses concentrated where the stucco is rigidly adhered to the face of the concrete footing or stemwall. If the wire lath in the original construction was not reliably connected to the studs, it is even possible for detachment to occur without earthquakes, and for essentially unattached panels to have existed for years without being noticed.

For additional information related to stucco, see Section 5.5.1 and Section 5.6.1 of the *General Guidelines*.

Plaster on Wood Lath

Plaster on wood lath is common in houses constructed prior to the 1940s. Its behavior during earthquakes is similar to that of weak stucco, including possible detachment of portions of the plaster from the lath during strong earthquakes (Figure 2-25). Plaster can provide significant bracing strength but often starts cracking and spalling at relatively low ground shaking and displacement levels.

For additional information related to plaster on wood lath, see Section 5.5.2 and Section 5.6.3 of the *General Guidelines*.

Figure 2-25 Detachment of plaster from wood lath (left) with destructive investigation (right) revealing that the wood framing had not sustained earthquake damage (photo credit: D. Bonowitz).

Plaster on Gypsum Lath

From the mid-1930s to the 1950s, interior wall finishes in houses transitioned from plaster on wood lath to gypsum wallboard. During that transition, many houses were built with plaster applied over gypsum lath panels. This wall finish system is sometimes called button board for the way the plaster is keyed to the gypsum panels with spaced round holes. Earthquake behavior at low displacement levels is similar to that of plaster on wood lath, including cracking. At higher displacement levels, behavior is more similar to that of gypsum wallboard (Figure 2-26).

For additional information related to plaster on gypsum lath, see Section 5.5.2 and Section 5.6.3 of the *General Guidelines*.

Figure 2-26 Earthquake-induced cracking and delamination of plaster on gypsum lath (photo credit: Exponent).

Gypsum Wallboard

Gypsum wallboard, also called drywall, is the most common interior wall finish material in houses built after the 1940s. Cracking of gypsum wallboard at corners of door and window openings is common in moderate-to-strong earthquakes, and cracking can also occur along taped joints between wallboard panels (Figure 2-27) and corner beads. Gypsum wallboard nails can work loose or pull out under earthquake shaking (or from other causes), leaving a pattern of raised nail heads discernible through the paint or taped finish. Where attached with screws, pullout is less likely, but with enough in-plane racking of the panel, the screws can gouge a slot in the drywall sheet not easily observed from the finished side. This "slotting" along the back sides of the panels reduces their stiffness in subsequent cycles of earthquake shaking. In more severe earthquakes, the panels may detach from the wall framing.

For additional information related to gypsum wallboard, see Section 5.5.3 and Section 5.6.2 of the *General Guidelines*.

Figure 2-27 Cracking of gypsum wallboard at corners of door opening caused by the 2018 Anchorage, Alaska earthquake (photo credit: Z. Stutts).

Wood Structural Panel, Particleboard, and Fiberboard Sheathing

Wood structural panel (WSP) sheathing is a general category that includes plywood (Figure 2-28) and oriented strand board. Particleboard and fiberboard sheathing, although less common in California, are grouped with WSP sheathing in these *Engineering Guidelines* because their use in seismic-force-resisting systems and their earthquake damage patterns are similar. A California house (or addition) built after about 1990 is likely to have a seismic-force-resisting system consisting of WSP-sheathed shear walls, although houses are still constructed without it. WSP sheathing is also found in retrofitted older houses, especially applied to the inside face of cripple wall studs.

The most common earthquake damage to WSP sheathing is associated with the fasteners (primarily nails in house construction), which are expected to yield (ideally), pull out or through panels, or fracture before the fields of the panels are damaged. In more extreme cases, damage associated with WSP sheathing can include splitting of the wood framing to which the panels are attached, in-plane rotation and uplift of the panels and their framing, or detachment of the panels from the wall framing. In the case of WSP-sheathed shear walls with high aspect ratios, damage may include rocking of the piers and associated damage to tie-down hardware at the ends of the shear walls, if present. Structurally significant damage to WSP sheathing is accompanied by conspicuous damage to the wall finishes covering the panels or applied to the opposite faces of the same wall lines.

In addition to the WSP sheathing damage patterns, particleboard or fiberboard sheathing can sustain earthquake damage in the form of crushing and tearing of the softer panel material around fasteners.

Figure 2-28 Wood structural panel (plywood) sheathing revealed by the removal of exterior finishes (photo credit: E. Mikailian).

Diagonal Lumber Sheathing

Prior to the mid-20th century, diagonal lumber sheathing (Figure 2-29) was used as a wall bracing system in many houses. Diagonal lumber sheathing typically consists of 1-inch thick boards laid diagonally across the wall framing and nailed to the studs and plates. Diagonal lumber sheathing is stiffer and stronger than horizontal lumber sheathing (discussed below). Earthquake damage to diagonal lumber sheathing includes splitting of the boards around nailed connections and, at higher shaking intensities, splitting of the wall framing to which the boards are attached.

Figure 2-29 Diagonal lumber sheathing revealed by the removal of exterior finishes (photo credit: D. Bonowitz).

Horizontal Lumber Sheathing

Horizontal lumber sheathing, particularly underlying the exterior wall finishes, is common in houses built before the mid-20th century (Figure 2-30). Behavior of horizontal lumber sheathing during earthquakes is similar to that of horizontal wood siding.

Figure 2-30 Horizontal lumber sheathing revealed by the removal of exterior finishes (photo credit: D. Bonowitz).

Framing

Conventional wood framing is more flexible than the wall sheathing and finish materials discussed above, so earthquake damage to the framing is almost always either preceded by damage to the sheathing or related to the load paths between the sheathing and the framing. Splitting of wood framing around sheathing fasteners can occur during earthquakes, particularly in the case of stronger sheathing materials, such as WSP or diagonal lumber sheathing. Separations between framing members can also occur due to rocking or sliding of wall piers.

In more extreme cases, earthquakes can cause wood stud walls to permanently rack in plane or lean out of plane. These conditions are generally associated with severe damage to, or detachment of, the wall's sheathing or brittle finish materials. Windows and doors will generally be inoperable along wall lines that experience significant racking or leaning.

Tie-downs and Other Load Path Elements

Tie-down hardware is sometimes found in newer houses with an engineered design, particularly at the ends of narrow shear walls (Figure 2-31). Similar hardware is used in combination with posts to provide load paths for overturning effects between shear walls in successive stories. Common anchorage systems include proprietary bracket-type tie-down devices, light-gauge metal straps, and threaded rods used in stacked shear wall systems. All these devices are intended to resist tension forces associated with uplift of the shear walls.

Figure 2-31 Tie-down at end of wood structural panel shear wall (photo credit: E. Mikailian).

Earthquake damage associated with tie-downs involves elongation or deformation of the devices themselves, or more commonly, damage to the elements to which they are attached, such as splitting of wood posts and spalling of concrete foundations. The force and deformation levels required to cause damage to these anchorage systems will typically result in widespread damage to more vulnerable elements of houses, such as brittle wall finishes.

Load path elements include the nails, screws, bolts, straps, blocking, and clips that connect the myriad framing members of houses. Earthquake damage patterns associated with separation between framing elements typically involve damage to such connections. Absent signs of significant separation or relative movement between framing elements, it is unlikely load path elements will have sustained damage. It is more likely that such framing separations occur because modern load path elements were not provided at all.

Gravity-Load-Carrying Elements

Although generally not a part of seismic-force-resisting systems, gravity-load-carrying elements such as posts (Figure 2-32) can experience residual drift or shifting due to earthquakes. Posts with inadequate connections at the tops or bottoms are more susceptible to these damage mechanisms. Recent shifting at post connections is typically accompanied by freshly exposed wood surfaces along the connections and distress to connection fasteners, if present.

Figure 2-32 Gravity-load-carrying post supporting a beam. The two members, which are toe-nailed together, are misaligned, but lack of an unpainted surface indicates no recent relative movement (photo credit: Exponent).

2.3.6 Other Seismic-Force-Resisting Elements

Portal Frames

Portal frames are a modern solution to the earthquake vulnerability associated with narrow wall piers around large openings, such as garage doors. The portal frame functions in a manner similar to a steel moment frame, with moment connections occurring between the top of the narrow wall piers and a header or other continuous beam at the top of the wall pier. The bottoms of the wall piers may or may not be intended to have moment fixity. The construction of portal frames is similar to that of engineered shear walls, except that in addition to tie-downs at the wall bases, moment fixity is created between the tops of the walls and beams above, often with either pairs of steel straps providing tension connections between the ends of wall piers and beams above or lapping of the wood structural panel sheathing over the faces of the beams with dense nail patterns. Portal frame provisions can be found in the prescriptive provisions of the *International Residential Code*, but portal frames can also have engineered designs.

Earthquake damage to portal frames can include concentrated damage to the sheathing or straps at the connections between the narrow wall piers and the beams (Figure 2-33). Concentrated damage to the sheathing and anchors may also occur at the bases of narrow wall piers, associated with rocking of the piers.

Figure 2-33 Concentrated earthquake damage to stucco at the top of a narrow wall segment adjacent to a garage door (photo credit: K. Cobeen).

Proprietary Shear Walls

Pre-engineered, manufactured wall panels may be used on new houses or be used for retrofitting existing houses (Figure 2-34). These proprietary systems are typically used where the available wall length is narrow, for example on either side of a wide garage door. With typically high shear capacity and high aspect ratios, proprietary wall panels require careful detailing of boundary elements and base attachments. Earthquake damage to these panels includes separation of the panels from the surrounding wood framing and damage to the tie-downs.

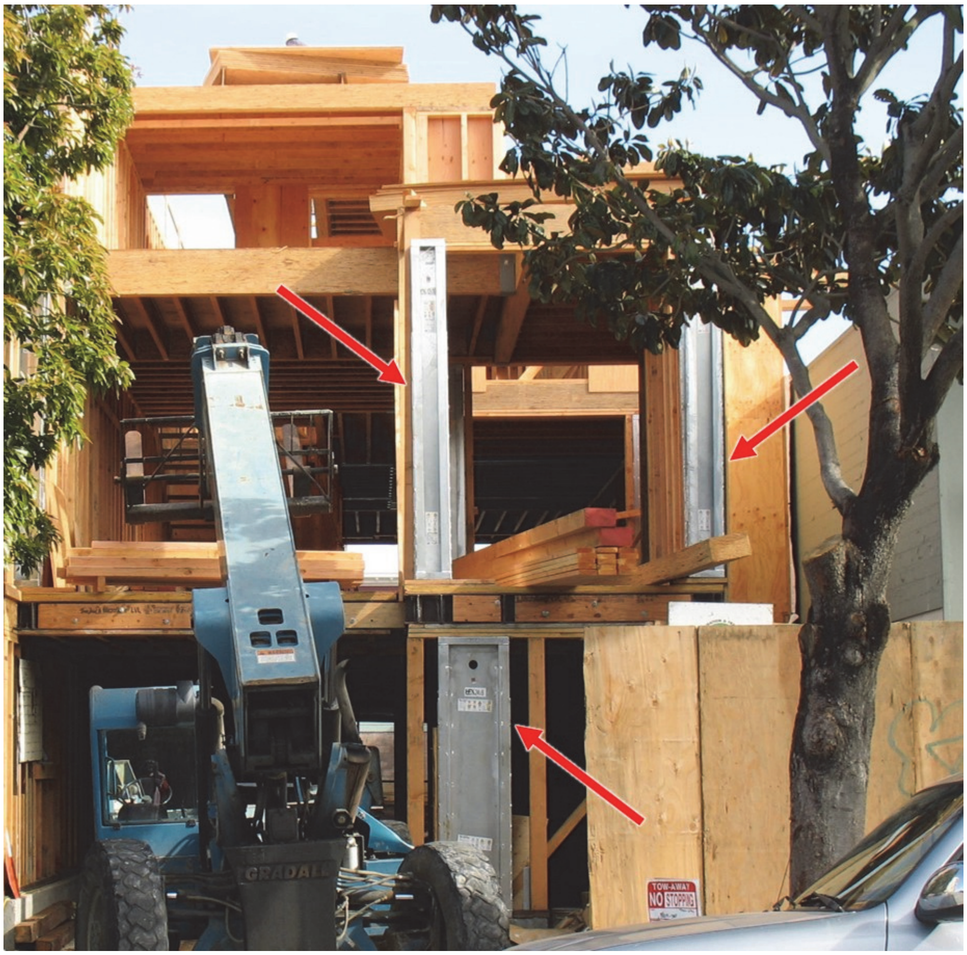


Figure 2-34 Proprietary shear walls at narrow wall piers between large window and door openings (photo credit: D. Bonowitz).

Hillside Houses: Diagonal Braces

Diagonal lumber bracing is found in some older hillside houses between the floor framing and foundations. (This is lumber specifically installed to provide bracing, as opposed to the let-in bracing that is sometimes used to square-up walls before sheathing is installed.) The bracing may be nailed or bolted to the tops and bottoms of posts, which support the gravity loads of the superstructures. The bracing is often ineffective in compression, so it is typically used in an X-pattern. Typical earthquake damage to diagonal lumber bracing includes splitting of the bracing at connections (Figure 2-35), or in the case of nailed connections, withdrawal of the fasteners from the framing to which the bracing is attached. Diagonal lumber bracing can be slightly bowed following earthquakes due to buckling of the bracing in compression, but such bowing typically does not affect the ability of the braces to resist tension loads during future earthquakes.

Steel rods can be used in lieu of diagonal lumber bracing. Typical earthquake damage to diagonal rod bracing includes elongation of the rods, as evidenced by slack and sagging rods following earthquakes. Turnbuckles are sometimes provided along the rods to tighten the rods during installation, and if so, slack rods can be retensioned following earthquakes. In extreme cases, the rods may fracture. Earthquake damage to the connections between the rods and framing members can also occur, such as fastener withdrawal and splitting of the framing members.

Figure 2-35 Damage at bolted connection of diagonal lumber bracing to post at hillside house. Damage at the connection includes splitting of the diagonal lumber brace and fracture of the post (photo credit: R. Gallagher).

Steel Moment Frames and Cantilever Columns

Like proprietary wall panels, structural steel seismic-force-resisting elements can be used for new houses and for retrofits, usually where the available wall lengths would not allow code-designed wood-frame elements. Due to their high shear capacities, these elements require engineered foundations and careful detailing at their attachments to wood-frame structures and to foundations.

Earthquake damage to steel moment frames and cantilever columns typically includes damage to the connections between the steel members and wood framing, including splitting or fracture of the wood framing. Damage may also occur at connections using steel anchor bolts embedded into the foundations. To provide adequate stiffness, these steel elements are often much stronger than typical earthquake design procedures would require, so the steel members themselves are unlikely to experience earthquake damage that reduces their strength before extensive damage has already occurred elsewhere in houses. If undersized, however, damage to the steel members can include bending or torsional deformation, weld fracture, or baseplate deformation or fracture.

2.3.7 Floors, Ceilings, and Roofs

Floor, Ceiling, and Roof Framing

Damage to floor, ceiling, or roof framing in earthquakes is rare and is typically associated with forces imparted by adjacent chimneys (Figure 2-36) or other heavy elements attached to walls, separations at framing discontinuities, or loss of vertical supports due to damaged foundations, posts, or wall framing. Significant damage to floor, ceiling, or roof framing will typically be accompanied by conspicuous damage to their finishes.

Figure 2-36 Damage to porch roof framing caused by chimney collapse (photo credit: R. Gallagher).

Many houses have conventionally braced roof framing that includes strongbacks to reduce rafter spans, kickers from the strongbacks to the tops of walls or ceiling braces, and kickers to provide gable end bracing. This framing has been known to come loose during earthquake shaking so that rafters or gable end walls are no longer properly braced.

For additional information related to floor, ceiling and roof framing, see Section 6.5 and Section 6.6 of the *General Guidelines*.

Floor Diaphragms

Structurally significant earthquake damage to floor diaphragms is rare. The most common damage patterns are associated with the failure of attachments of the floor diaphragms to foundations and supporting walls. In rare instances, where the floor sheathing material is very weak, floor diaphragms could be subject to residual racking or distortion. In hillside houses with inadequate stiffness at the downhill end, the main floor diaphragm can be prone to in-plane racking or torsion, but this damage would be preceded by damage to other elements.

For additional information related to floor diaphragms, see Section 6.5.3 and Section 6.6.3 of the *General Guidelines*.

Ceiling Diaphragms

Like floor diaphragms, structurally significant earthquake damage to ceiling diaphragms is rare. Cracking of brittle ceiling finishes, such as gypsum wallboard and plaster, can occur at relatively low intensity shaking but is usually not structurally significant (Figure 2-37). Damage patterns to these finishes are similar to those of similar materials used as wall finishes. Damage to ceiling diaphragms may also occur where the ceiling finishes are continuous across discontinuous framing, such as at split levels, stairwells, or where the joist directions change. In one-story houses or in the uppermost story of multi-story houses, ceiling finishes typically serve as the diaphragm sheathing for ceilings, except in cases of attics with floor systems.

For additional information related to ceiling diaphragms, see Section 6.5.2 and Section 6.6.2 of the *General Guidelines*.

Figure 2-37 Cracking of ceiling finishes caused by the 2003 San Simeon, California earthquake (photo credit: Exponent).

Roof Diaphragms

Structurally significant earthquake damage to roof diaphragms sheathed with WSP or diagonal lumber sheathing is rare. Roof diaphragms sheathed with spaced horizontal boards supporting clay or concrete roof tiles can be prone to in-plane racking or related collapse. Damage to roof diaphragms can also occur at framing discontinuities.

For additional background information related to roof diaphragms, see Section 6.5.1 and Section 6.6.1 of the *General Guidelines*.

2.3.8 Fireplaces and Chimneys

Chimneys and Fireboxes

Masonry chimneys are among the most vulnerable elements of houses. Unreinforced chimneys are vulnerable to cracking near rooflines (Figure 2-38), sometimes breaking away and falling. If the top portion of a chimney falls onto the roof, it can cause damage to the roofing and roof framing. Reinforced chimneys can separate from adjacent walls (Figure 2-39) and exhibit residual outward leaning following earthquakes. Many chimneys were built with steel straps embedded in the masonry and attached to floor or roof framing. Where inadequately connected, the straps can pull out, damaging the framing members or roof sheathing.

Figure 2-38 Earthquake-induced fractured chimney near the roof line. The brace between the chimney and roof, visible behind the chimney, was not adequate to prevent the damage (photo credit: Exponent).

Figure 2-39 Separation of chimney from house caused by the 2014 La Habra, California earthquake. The separation was relatively minor, and the chimney remained attached to the house (photo credit: Exponent).

After a damaging earthquake in an area, an inspection of the flue by a chimney consultant may be warranted, since damage to the flue could constitute a fire hazard. Issues related to chimney flues are outside the scope of this document.

For additional information related to chimneys and fireboxes, see Section 7.5.1, Section 7.5.2, Section 7.6.1 and Section 7.6.2 of the *General Guidelines*.

Interior Fireplace Surrounds

Interior fireplace surrounds, typically consisting of brick, tile, or stone veneers, are often poorly secured to the wall framing or finishes. Such veneers are prone to cracking and detaching from walls, creating potential falling hazards and locations of likely damage.

For additional information related to interior fireplace surrounds, see Section 7.5.3 and Section 7.6.3 of the *General Guidelines*.

2.3.9 Nonstructural Components

MEP Component Bracing and Anchorage

Common MEP components of houses include water heaters, furnaces, air conditioning units, and pool or spa equipment, as well as their associated piping. Many of these MEP components in modern houses are anchored or braced; similar components in older houses are frequently unanchored and unbraced. Water heaters in older houses are likely to have been strapped to the adjacent wall if the water heaters have been recently replaced or if the properties have been recently purchased; however, poorly installed or anchored water heater straps are common (Figure 2-40).

Earthquakes can result in shifting or toppling of inadequately anchored and braced MEP components, possibly resulting in damage to anchorage or bracing that may be present, as well damage to building elements to which the anchorage or bracing is attached. When mounted on the rooftop or otherwise elevated, displaced MEP equipment can constitute falling hazards. Damage to the actual MEP components is outside the scope of this document.

Figure 2-40 Inadequately strapped water heater (photo credit: D. Bonowitz).

Architectural Component Bracing and Anchorage

Houses typically contain a wide variety of architectural components that can become damaged or dislodged during earthquakes. Common components include clay or concrete roof tiles and brick or stone veneer. Roof tiles can become loose and present falling hazards in the event of aftershocks (Figure 2-41). Brick or stone veneer that is inadequately anchored to walls may separate from the walls or collapse.

Figure 2-41 Displaced roof tiles caused by the 2019 Ridgecrest, California earthquake (photo credit: M. Ziemer).

2.3.10 Appurtenances

Earthquake damage tends to concentrate where parts of a house with different construction abut or attach to each other. This often occurs at appurtenances, such as exterior decks, porch roofs, carports, or patio covers, where the damage is likely to include cracked finishes, damage to ledgers or fasteners, and residual separations. An appurtenance with no seismic-force-resisting system of its own may be vulnerable to collapse if its attachment to the main structure is damaged (Figure 2-42). Damage to an appurtenance, such as a deck at an entry door, can block ingress and egress to a house

Figure 2-42 Earthquake-caused detachment of the carport from the house wall, resulting in collapse (photo credit: California Office of Emergency Services).

2.4 Intensity of Ground Shaking and Damage Potential

Earthquake damage to houses is directly related to the intensity of ground shaking at sites. The intensity of shaking can be measured in many ways, including the Modified Mercalli Intensity (MMI) scale, which has been the most widely used intensity scale in the United States since the 1930s. The scale is based on the observed local effects of ground motions on people, building contents, buildings, and the environment. The scale ranges from I to XII, with I being imperceptible and XII being total destruction of the built environment.

Each level of intensity is characterized by certain effects (e.g., many small objects overturned and fallen) that are commonly observed at that intensity level. The same effect can occur less frequently or less strongly at lower intensity levels and can occur more frequently and more strongly at higher intensity levels. For example, at MMI VI, overturned furniture occurs in many instances; at MMI V and below, occurrences of overturned furniture are rare; whereas, at MMI VII and above, occurrence of overturned and possibly damaged furniture is common. In areas where the intensity of shaking is below MMI V, damage to buildings or contents is extremely unlikely. At MMI V, some damage to contents but no damage to buildings would be expected. Minor damage to some buildings occurs at intensity VI. Progressively more building and contents damage occurs at MMI VII through IX. MMI X through XII are generally associated with permanent ground deformations and very severe damage to buildings and contents.

ShakeMaps, which are generated by the U.S. Geological Survey within minutes of major earthquakes in the United States and are published on the agency's website, provide an automated surrogate of MMI based on seismic instrument recordings (Instrumental Intensity). ShakeMaps provide the technical consultant historical context regarding the severity of earthquake damage expected at sites and can be used to relate the shaking and content disturbances experienced by building occupants to building damage.

CHAPTER 3

Damage Investigation: Structural

3.1 Introduction

This chapter provides a systematic procedure for finding, classifying, and documenting earthquake damage to the structural elements of a house. It presumes that the damage investigation process will rely on non-destructive visual and tactile observation by a qualified consultant (see Section 1.1), supplemented by interviews of individuals who witnessed the damage or are familiar with the pre-earthquake condition, reports or observations of damage (or the lack of it) at nearby sites, ground motion data, and, where needed, destructive investigation. The chapter presumes the investigation will include reasonable efforts to access all areas of the house, including confined spaces, such as attics and crawlspaces, with appropriate tools and safety procedures. Means and methods of investigation are left to the structural consultant.

The procedures in this chapter are suitable for any of the investigation contexts discussed in Chapter 1. Where the context calls for only a partial investigation, some sections of this chapter will not be needed and the elements in question should be noted as not investigated. Otherwise, the chapter is written to be used in a systematic way presuming the intent of the investigation is to produce a comprehensive report that covers the entire house. To support that intent, this chapter's sections are numbered to align with the background discussions in Chapter 2 and with the repair recommendations in Chapter 4. Some terms used in this chapter are defined and discussed further in the Glossary.

3.1.1 Organization

The chapter is organized into sections covering the main elements of a typical house. Section 3.2 through Section 3.10 contain the following contents:

Scope. This material offers guidance on how to apply the checklist to typical elements of a house.

Indicators. This material, presented in a table, offers guidance for the investigation of concealed structural elements. Indicators suggest the possibility of related earthquake damage, but the list of indicators is not a list of possible damage patterns for the structural element being investigated. Where the element in question is directly observable, indicators are not needed. Where the element is concealed, however, damage can often be reasonably suspected or ruled out based on the condition of observable connected or adjacent elements and nonstructural finishes. The indicators are damage patterns to those related elements and finishes. The presence of one or more indicators is often associated with **Suspected** damage, discussed in Section 3.1.2, to the structural element in question, and it can be the justification for destructive investigation, discussed in Section 3.1.4.

Checklist. The main substance of each section is the investigation checklist of element types, one or more of which may be present at a given house. For each element type present, the structural consultant records whether damage, from the earthquake or not, is observed. The purpose of the checklists is to help ensure complete and consistent investigations. While the checklist format lends itself to use as a field tool, the version provided here is expected to be useful only as a way of categorizing the investigation findings, summarizing the conclusions, and coordinating with the repair recommendations in Chapter 4. As such, each checklist records only the most basic information: which element types are present at the house, which categories of damage were observed, and which areas were not investigated. A more complete field tool could be based on these basic checklists, but, as discussed under Section 3.1.3, it might also provide space for recording notes, explanations, sketches, and quantities to support a repair scope or cost, as well as separate rows for different subsets of similar elements, such as the perimeter walls on each side of the house.

3.1.2 Checklist Content

The first two columns (**Check if Present** and **Element Type**) of each investigation checklist establish the scope of the investigation as it applies to the house in question. The next three columns (**Yes**, **Suspected**, and **No**) provide three possible responses related to earthquake damage for each element type present. The last two columns (**Non-EQ Damage** and **Not Investigated**) are used, respectively, to indicate that observed damage is judged to have a cause other than the earthquake or has not been significantly worsened by the earthquake or to indicate that an investigation of the element type has not been completed. Each of these possible checklist responses is described in more detail in this subsection. Each checklist also includes a series of **Repair Threshold Reminders**, which are prompts for damage thresholds that, if observed, should be documented. These are discussed in more detail in Section 3.1.3.

Check if Present This column is used to indicate which element types exist at the house. For basic elements present in every house, such as the ground supporting the house (Section 3.2) or wall framing (Section 3.5), the box will always be checked. In most checklists, however, this column is used to indicate which of several common element types are actually present. This column thus records the basic scope of the investigation and offers a useful visual aide for scanning the completed checklist.

Some element types, such as tie-downs or wood sheathing (Section 3.5), are typically concealed, but their presence should not be assumed. For these, the checklists include check boxes (**Assumed**, **Confirmed**) in the Element Type column to indicate whether the existence of the element was actually confirmed or only assumed.

Element Type Each checklist row corresponds to one type of common element. As noted above, for some element types, this column includes check boxes to note whether typically concealed elements are confirmed or assumed.

This column also includes **Repair Threshold Reminders**, discussed below under Section 3.1.3.

Yes Where the earthquake is believed to have caused damage of some sort to the structural element in question, the appropriate response is **Yes**. Even if the damage is structurally insignificant or the appropriate repair is already covered by the *General Guidelines*, any observed earthquake damage should still be recorded with a **Yes** response.

Earthquake damage to nonstructural finishes, nonstructural components, or contents adjacent or connected to the structural element in question is not by itself considered earthquake damage to the structural element. As described, such damage might indicate damage to a concealed structural element, in which case **Suspected** might be the appropriate response. Otherwise, the nonstructural damage can be documented for completeness, but the appropriate response regarding the structural element in question is most often **No**.

Damage from a prior earthquake is not counted as earthquake damage for purposes of the present investigation. The appropriate response to this damage is **No**, with an option to also check the box for **Non-EQ Damage**.

Pre-existing damage believed to have been worsened by the earthquake should only be counted as earthquake damage when the earthquake effect has made the damage categorically different, such that Chapter 4 would call for a different type of repair for the pre-earthquake and post-earthquake conditions. This determination calls for engineering judgment regarding the relative contribution of the earthquake and non-earthquake causes.

For example, consider a vertical crack in a concrete stemwall up to 1/16-inch wide with no out-of-plane offset. Such a crack is characteristic of normal concrete shrinkage and would reasonably be categorized as **Non-EQ Damage**. Even if it were considered earthquake damage, Table 4-2 indicates no structural repair.

- If upon investigation, a crack is found with characteristics of shrinkage cracks but with a width still less than 1/8 inch, it might be argued that the earthquake widened a pre-existing shrinkage crack by a factor of two or more. Even so, as Table 4-2 shows, the wider crack would still require no structural repair. Since the repair (or lack of repair, in this case) would be the same for the pre-earthquake and post-earthquake conditions, the widening would be discounted, and the crack would be categorized as **Non-EQ Damage**.
- If a similar crack is found with a width up to 1/2 inch, Table 4-2 would call for repair by epoxy injection. Assuming the widening can be reasonably ascribed to the earthquake, this crack would be categorized as earthquake damage even though the concrete was already cracked from shrinkage.

- If a 1/2-inch crack is found, but evidence (such as debris or efflorescence patterns) indicates that it was already 1/8-inch to 3/8-inch wide before the earthquake, then by Table 4-2, the now-1/2-inch crack is not categorically different from its pre-earthquake condition, so this crack would be categorized as **Non-EQ Damage**.

Suspected Where earthquake damage to the structural element is not directly observable but is suspected based on observable evidence, **Suspected** is an appropriate response. (See the discussion of Indicators above.) **Yes** and **No** can also be appropriate where damage to the concealed element is reasonably assumed or ruled out, respectively, subject to the judgment of the structural consultant. **Suspected** or assumed damage to concealed elements should always be supported by evidence; **Suspected** is not the appropriate response just because the structural element is concealed.

Examples of suspected damage include damage to wood sheathing or framing concealed by wall finishes, damage to a masonry chimney embedded in a wall, and damage to a concrete slab under carpet or flooring. Each section below offers guidance on indicators of damage to concealed elements, but whether damage to the structural element in question is suspected is left to the judgment of the structural consultant.

Suspected damage may be resolved with destructive investigation where needed; see Section 3.1.4.

No Where earthquake damage can reasonably be ruled out, the appropriate response is **No**. Earthquake damage should not be presumed simply because the site experienced strong ground motion or because a neighboring building sustained damage. On the contrary, it is almost always reasonable to rule out earthquake damage in a house where no fresh damage is directly observable in exposed structural elements or in nonstructural finishes that serve as Indicators. That said, cases of very light damage are less likely to need investigation by a structural consultant.

See the discussion at **Yes** regarding non-earthquake damage worsened by the earthquake.

Often, only one of these three responses (**Yes**, **Suspected**, or **No**) will be selected for each element type. At the discretion of the structural consultant, more than one response may be selected to distinguish between different locations within the building. For example, earthquake damage might be directly observed on one side of the house but ruled out or only suspected on the other sides. Documentation should clarify the intended meaning of each response.

Non-EQ Damage Identification of non-earthquake damage is a secondary priority and checking the box for **Non-EQ Damage** should be considered optional in most cases. This column is intended to help the structural consultant record a conclusion regarding a pattern of damage that might be mistaken for earthquake damage or is otherwise in dispute. Since the *Engineering Guidelines* are not designed to yield a complete list of all non-earthquake damage, the checklist column for non-earthquake damage is not in the same Yes/Suspected/No format as the columns for earthquake damage.

Checking the box for **Non-EQ Damage** means that observed damage is judged to have a cause other than the earthquake or has not been significantly worsened by the earthquake; see the discussion at **Yes** regarding non-earthquake damage worsened by the earthquake. Damage from a prior earthquake is also considered non-earthquake damage for purposes of the present investigation. Earthquake and non-earthquake damage can both be present in the same element.

Not Investigated For earthquake damage, where none of the three responses is justified due to a limited investigation, the appropriate response is to check the box under **Not Investigated**. This box is intended to record cases where the normal investigation scope could not be completed, typically due to unsafe or inadequate access. Checking the **Not Investigated** box can also be a way of indicating that the element in question was outside the scope of an intentionally limited investigation. Documentation should explain the reason for a **Not Investigated** response.

Where destructive investigation is needed to resolve a question of suspected damage (see Section 3.1.4), the appropriate response is almost always **Suspected**, not **Not Investigated**.

3.1.3 Documentation

Observed damage in any category should be documented as needed to support repair decisions described in Chapter 4 and to inform the investigation report outlined in Chapter 8. Beyond this general instruction, these *Engineering Guidelines* do not prescribe documentation tools, methods, or content, but typical investigations involve written, recorded, or photographic documentation of:

- observed damage patterns and their locations within the house or the site, perhaps on a sketched plan;
- qualitative and quantitative damage descriptions, including crack widths, deformations or deflections relative to the pre-earthquake condition, and areas or numbers of damaged and undamaged elements;
- explanations or other supporting evidence for damage categorizations, including damage descriptions for Indicator damage patterns; and
- construction quantities that might be needed to inform a repair scope or cost.

Since the damage patterns and descriptions determine the repair recommendations in Chapter 4, it is especially useful to document whether observed damage exceeds one or more of the threshold values used there. To facilitate this, the checklists include **Repair Threshold Reminders** coordinated with the Chapter 4 tables. These reminders are not instructions about what to investigate, they are not a complete list of possible damage patterns for each element type, and they are not a complete list of the damage descriptions used in Chapter 4. They are only the thresholds that determine, in part, the repair recommendations in the *Engineering Guidelines*, and they are provided in the checklists for the convenience of the structural consultant. Users are referred to Chapter 4 for more complete descriptions of threshold damage patterns.

Where damage is suspected but not confirmed, an interim report calling for additional destructive investigation might be appropriate. Otherwise, the procedure can continue to Chapter 4 for repair recommendations based on one or more assumed conditions, and the decision to perform additional or destructive investigation can be made based on the potential significance to the report's overall conclusions.

3.1.4 Destructive Investigation

This chapter presumes an investigation based primarily on non-destructive observation, generally visual or tactile. Where damage to a concealed structural element is suspected based on Indicators or other evidence, destructive investigation is often appropriate. Destructive investigation is not permanently destructive; for purposes of the *Engineering Guidelines*, destructive investigation means any investigation other than direct visual or tactile observation. Examples of destructive investigation include the selective pulling of carpets to assess evidence of slab damage and selective removal of wall finishes, such as stucco or plaster, as needed to assess evidence of damage to wall framing or sheathing.

Destructive investigation might also be used to confirm assumptions that significantly affect the report's overall conclusions. For example, Section 3.5 notes that the presence of wood structural panel sheathing should not be assumed; if the nature or extent of repairs to stucco, for example, would be significantly different if the stucco were known to be an architectural finish only, then limited destructive investigation might make sense to confirm the actual sheathing material and details. In general, however, it is not necessary to perform destructive investigation just to confirm the existing construction.

In all cases, destructive investigation should be incremental, to limit disruption to the occupants. A small observation area or opening is often sufficient; it can be enlarged as needed to confirm or rule out suspected damage. Means and methods of destructive investigation are left to the structural consultant.

In the insured context, destructive investigation might require advance approvals. In almost all cases, it requires special tools, personnel, or procedures (including, potentially, hazardous materials abatement) not normally provided at the initial site visit. For these reasons, destructive investigation is rarely performed during an initial investigation. Instead, destructive investigation can be deferred to later site visits or until it can be combined with the repair work.

3.2 Ground

3.2.1 Scope

Earthquake-induced permanent ground deformation is covered in Chapter 5 through Chapter 7 and is generally outside the scope of Chapter 3. If a geotechnical consultant has not yet been consulted, the structural investigation should include a high-level review for indicators of earthquake-induced permanent ground deformation; see Table 3-1. Table 3-1 is different from the other tables of indicators in this chapter because it is not limited to concealed damage. Rather, its purpose is to provide guidance to the structural consultant about when the use of a geotechnical consultant should be considered.

For similar reasons, the investigation checklist for Section 3.2 is different from the other investigation checklists. While Chapter 3 is intended as guidance for the structural consultant, the determination of whether observed permanent ground deformation was caused by the earthquake is often made by a geotechnical consultant. Thus, the investigation checklist is not a thorough listing of element types but merely a means of recording general observations.

- Where permanent ground deformation is suggested by one or more indicators, Suspected will often be the best response.
- Where earthquake-induced permanent ground deformation is obvious, Yes can also be an appropriate response, and a geotechnical consultant may be consulted as needed to specify appropriate repairs. In particular, even if a geotechnical consultant is not needed to identify or categorize observable damage, one might be used where the remaining capacity or stability of the ground, as needed to support the house, is unknown, or where the nature or extent of necessary building repairs might require repair of permanent ground deformation itself.
- Where permanent ground deformation is neither observed nor suspected, No is an appropriate response. Thus, the *Engineering Guidelines* acknowledge that a damage investigation can sometimes be completed without the need for a geotechnical consultant.
- Where permanent ground deformation unrelated to the earthquake is observed, the consultant may check the Non-EQ Damage box. A geotechnical consultant may then be consulted as needed to confirm the cause of the damage.

3.2.2 Indicators and Checklist

Table 3-1 Indicators of Permanent Ground Deformation

Element Being Investigated	*Indicator*
Ground supporting the house or ground outside the house footprint	• Observable permanent ground deformation on or adjacent to the site: o Surface fault rupture o Liquefaction o Lateral spreading o Landslide or slump indicative of a landslide o Rock fall either within or onto the site o Fill settlement o Ground cracking • Flatwork damage, including at the curb or street • Utility line damage • Retaining wall damage • Swimming pool crack, leak, or out-of-levelness • Foundation damage (see Section 3.3)

Investigation Checklist 3-1 Permanent Ground Deformation

Check if Present	*Element Type*	*Earthquake Damage*			*Non-EQ Damage*	*Not Investigated*
		Yes	*Suspected*	*No*		
☐	Ground supporting the house	☐	☐	☐	☐	☐
☐	Ground outside the house footprint	☐	☐	☐	☐	☐

3.3 Foundations, Slabs-on-Grade, and Basement Walls

3.3.1 Scope

This section covers the most common foundation types, as well as basement walls.

- The slabs-on-grade type includes concrete slabs with and without a thickened edge or edge footing cast monolithically. Where an edge footing is cast separately, both the slab-on-grade and the concrete footing subsections apply.
- The slabs-on-grade type may be used for exterior flatwork, such as patios.
- Masonry footings or stemwalls include both reinforced and unreinforced masonry.
- Where masonry stemwalls are supported on concrete footings, consider both the concrete and masonry element types.
- Retaining walls that are extensions of basement walls may be addressed in this section as basement walls. Retaining walls outside the house footprint are considered in Chapter 5 through Chapter 7.
- The posts and piers foundation type includes shallow piers only. For drilled piers or caissons, the structural consultant should develop a reasonable investigation protocol.

- The slabs-on-grade element type may be used for post-tensioned slabs, but if significant damage is found, the structural consultant should also consider the possibility of tendon damage. Post-tensioned slabs are often indicated by tendon pockets along the side edge of the slab or by a "Do not cut or core" warning stamped into the slab surface.

3.3.2 Indicators and Checklist

Table 3-2 Indicators of Damage to Concealed Foundations, Slabs-on-Grade, and Basement Walls

Element Being Investigated	*Indicator*
Slabs-on-grade	• Cracking or widened joints in brittle floor finishes (e.g., tile, hardwood) • Distortion of pliable floor finishes (e.g., carpet, vinyl)
Footings or stemwalls, concrete	• Damage to finish adhered to or covering the concrete • Out-of-plane leaning of cripple wall studs, indicating footing rotation
Footings or stemwalls, masonry	• Damage to finish or encasement adhered to or covering the masonry (Encasement refers to a layer of concrete or mortar applied to the top and sides of an unreinforced masonry element as a prior repair or retrofit.) • At footing, out-of-plane leaning of cripple wall studs, indicating footing rotation
Basement walls, concrete	• Damage to finish adhered to or covering the concrete • Mass movement or settlement of retained soil • For partial-height wall, out-of-plumb wood studs at upper portion of basement story
Basement walls, masonry	• Damage to finish adhered to or covering the masonry • Mass movement or settlement of retained soil • For partial-height wall, out-of-plumb wood studs at upper portion of basement story

Investigation Checklist 3-2 Foundations, Slabs-on-Grade, and Basement Walls

Check if Present	*Element Type*	*Earthquake Damage*			*Non-EQ Damage*	*Not Investigated*
		Yes	*Suspected*	*No*		
☐	Slabs-on-grade *Repair threshold reminders:* • *Crack 1/8 inch, 1/2 inch* • *Crack offset 1/16 inch* • *Drop or slope 1/2 inch in 8 feet, 2 inches in 8 feet* • *Loss of bearing*	☐	☐	☐	☐	☐
☐	Footings or stemwalls, concrete *Repair threshold reminders:* • *Crack 1/8 inch, 1/2 inch* • *Crack offset 1/16 inch, 1 inch* • *Drop or slope 1/2 inch in 8 feet, 2 inches in 8 feet* • *Tilt or lean 1/4 inch per foot* • *Loss of bearing*	☐	☐	☐	☐	☐

Investigation Checklist 3-2 Foundations, Slabs-on-Grade, and Basement Walls (continued)

Check if Present	Element Type	Earthquake Damage: Yes	Earthquake Damage: Suspected	Earthquake Damage: No	Non-EQ Damage	Not Investigated
☐	Footings or stemwalls, masonry *Repair threshold reminders:* • *Crack in brick or masonry unit 1/8 inch, 1/2 inch* • *Crack in mortar joint 1/8 inch, 1/2 inch* • *Crack offset 1/16 inch* • *Drop or slope 1/2 inch in 8 feet, 2 inches in 8 feet* • *Tilt or lean 1/4 inch per foot* • *Loss of bearing*	☐	☐	☐	☐	☐
☐	Posts and piers *Repair threshold reminders:* • *Post lean 1/2 inch total over post height* • *Post lean 1/4 inch per foot over height* • *Floor drop 1/16 inch per foot and 1/4 inch total in 8 feet* • *Floor drop 1/4 inch per foot and 2 inches total in 8 feet* • *Deep pier shift 2 inches*	☐	☐	☐	☐	☐
☐	Basement walls, concrete *Repair threshold reminders:* • *Crack 1/8 inch, 1/2 inch* • *Crack offset 1/16 inch* • *Drop or slope 1/2 inch in 8 feet, 2 inches in 8 feet* • *Tilt or lean 1/4 inch per foot of height* • *Loss of bearing*	☐	☐	☐	☐	☐
☐	Basement walls, masonry *Repair threshold reminders:* • *Crack 1/8 inch, 1/2 inch* • *Crack offset 1/16 inch* • *Drop or slope 1/2 inch in 8 feet, 2 inches in 8 feet* • *Tilt or lean 1/4 inch per foot of height* • *Loss of bearing*	☐	☐	☐	☐	☐

3.4 Sill Plates and Anchorage

3.4.1 Scope

This section covers the interface between the foundation, assumed to be concrete or masonry, and the walls above, assumed to be wood framed.

- Post-installed anchor bolts or plate connectors are indicative of a prior retrofit.
- For a hillside house, the framing to foundation connection can involve any number of details at the uphill end. At this critical location (see Section 2.2.3), the connection is often not a conventional wood sill plate with anchor bolts but can include steel hardware, bolted wood ledgers, or other elements. Consider all of the relevant element types.
- Tie-down hardware and anchorage is covered in Section 3.5.

3.4.2 Indicators and Checklist

Table 3-3 Indicators of Damage to Concealed Sill Plates and Anchorage

Element Being Investigated	*Indicator*
Wood sill plates	• Permanent movement of wood-frame wall relative to the top of foundation
Steel cast-in-place anchor bolts	• Permanent movement of wood-frame wall relative to the top of foundation • Fractured or dislocated sill plate
Steel post-installed anchor bolts	• Permanent movement of wood-frame wall relative to the top of foundation • Fractured or dislocated sill plate
Steel post-installed plate connectors	• Permanent movement of wood-frame wall relative to the top of foundation • Fractured or dislocated sill plate
Hillside houses: framing-to-foundation connections	• Leaning or racking of wood-frame side walls of underfloor area • At uphill side, widened gap between exterior flatwork and house

Investigation Checklist 3-3 Sill Plates and Anchorage

Check if Present	*Element Type*	*Earthquake Damage: Yes*	*Earthquake Damage: Suspected*	*Earthquake Damage: No*	*Non-EQ Damage*	*Not Investigated*
☐	Wood sill plates *Repair threshold reminders:* • *Slide 1/4 inch* • *Fracture at multiple adjacent bolts* • *Splitting away from bolts* • *Splitting of retrofit block* • *Crushing 1/8 inch*	☐	☐	☐	☐	☐
☐	Steel cast-in-place anchor bolts *Repair threshold reminders:* • *15-degree angle of bolt from vertical* • *Gap 1/4 inch between bolt shank and sill plate* • *Spall 1-inch deep at anchor bolt*	☐	☐	☐	☐	☐
☐	Steel post-installed anchor bolts *Repair threshold reminders:* • *15-degree angle of bolt from vertical* • *Gap 1/4 inch between bolt shank and sill plate* • *Spall 1/2-inch deep at anchor bolt*	☐	☐	☐	☐	☐
☐	Steel post-installed plate connectors *Repair threshold reminders:* • *15-degree angle of bolt from vertical* • *Gap 1/4 inch between bolt shank and sill plate* • *Splitting of sill plate with 20% capacity loss in fasteners* • *Spall 1/2-inch deep at anchor bolt*	☐	☐	☐	☐	☐
☐	Hillside houses: framing-to-foundation connection *Repair threshold reminders:* • *At uphill connection, shift 1/2 inch of superstructure relative to foundation*	☐	☐	☐	☐	☐

3.5 Wood-Frame Walls

3.5.1 Scope

This section covers the house's wood-frame seismic-force-resisting system (SFRS), comprising wood studs and common wall sheathing and finish types. Other types of seismic-force-resisting systems are covered in Section 3.6. This section also covers posts and wood-frame vertical elements expected to carry only gravity loads. Floor, ceiling, and roof framing is covered in Section 3.7.

Wood-frame walls include many concealed elements, including wood sheathing under finishes. As part of the initial investigation process, an attempt should be made to confirm whether sheathing is present. Often this can be done by looking in unfinished areas, such as attics, crawl spaces, closets, or the back of utility access panels. Destructive investigation to confirm the presence of sheathing during the initial investigation, however, is neither necessary nor recommended. Rather, if the presence of sheathing cannot be confirmed or reliably assumed based on the age of the house, direct observation, or construction documents, the default assumption for the initial investigation should be that this sheathing is *not* present. For this reason, the checklist includes check boxes to indicate whether the element type in question was actually confirmed or only assumed.

These *Engineering Guidelines* do not distinguish between wood-frame walls intended (or expected) to resist lateral forces and nominal wood-frame partitions that contribute unintended lateral strength and stiffness. Whether a wood-frame wall or partition contributes significantly to a house's lateral strength and stiffness is a function of the wall's length, openings, construction (sheathing or finishes), and load path connections to floor and foundation elements. Because it is often difficult to identify the sheathing and details from an initial visual investigation, and because the repair procedures for any observed damage are similar, the document treats all wood-frame walls and partitions the same way.

- Cripple walls, whether unbraced or retrofitted, should be addressed as wood-frame walls, considering all applicable sheathing or siding types, framing, tie-downs, and other load-path elements. See also Section 2.2.2 for a discussion of configuration issues.
- For a hillside house, the walls of the underfloor area, whether unbraced or retrofitted, should be addressed as wood-frame walls, considering all applicable sheathing or siding types, framing, tie-downs, and other load-path elements. See also Section 2.2.3 for a discussion of configuration issues. Diagonal bracing of the underfloor area is covered in Section 3.6.
- The element checklist includes three types of structural sheathing, as well as six types that provide strength and stiffness even though they are commonly considered finish materials. *International Residential Code* (IRC) Table R602.10.4 (or *International Building Code* (IBC) Table 2308.6.3(1) (ICC, 2018c)) uses different designations and includes still other bracing types. The element checklist types below relate to, and may be applied to, these IRC bracing types as follows:
    - The diagonal lumber sheathing type may be used for IRC diagonal wood boards (DWB).
    - The plywood panel siding type may be used for IRC hardboard panel siding (HPS).
    - The stucco type may be used for (and is identical to) IRC Portland cement plaster (PCP).

- The wood structural panel (WSP) type may be used for IRC alternate braced wall panels (ABW). Proprietary shear wall elements similar to code-prescribed ABW systems are covered in Section 3.6. (See also the discussion of narrow wall piers in Section 2.2.4.)
- IRC portal frame at garage (PFG) and IRC portal frame with hold-downs (PFH) are covered in Section 3.6.
- IRC let-in bracing (LIB) is not considered an adequate SFRS, so any damage to existing let-in bracing should be considered structurally insignificant; if damage to let-in bracing causes damage to the wood studs, that damage should be considered with the framing type.

* Siding made of wood shingles, vinyl, aluminum, lap siding, or other materials or details not specified in the checklist should be considered nonstructural. Unless there is evidence that they provide significant strength and stiffness to the existing structure, these siding types are outside the scope of this document. For example, typical lap siding should be considered nonstructural, but shiplap siding (see the Glossary) provides structural strength and stiffness and should be considered a type of horizontal wood siding for the purposes of applying the checklist.
* Framing includes the typical stud framing (platform or balloon framed) as well as headers, trimmers, and other framing members around openings; sole plates and rim joists at floors above grade; blocking or nominal construction bracing; and posts, corner studs, or other wood members built into the wall line.
* Tie-downs include the device hardware (typically light-gauge steel), the anchor bolt or rod, and fasteners to the wood framing. Other load-path elements include straps, clips, and other hardware (typically steel) within the wall line or connecting a wall to floor, ceiling, or roof framing.
* Gravity-load-carrying elements include posts or piers of any material, interior stair framing, as well as wood-stud framing that carries only gravity load. Posts can be within the interior of the house (or attached garage), where they typically support a long girder at midspan, or at the house perimeter, where they typically support a porch roof or an overhanging upper story.
  - Foundation posts within a crawl space or underfloor area are covered in Section 3.3.
  - Posts with diagonal bracing at the downhill end of a hillside house are covered in Section 3.6.

3.5.2 Indicators and Checklist

Because wood-frame walls comprise a number of elements, only some of which are normally concealed, indicators for wood-frame walls often involve damage to one element indicating a particular damage pattern in a different element within the same wall.

Table 3-4 Indicators of Damage to Concealed Wood-Frame Wall Elements

Element Being Investigated	*Indicator*
Any siding type included in the investigation checklist	• Extensive damage to architectural finishes (e.g., wallpaper, paneling, veneer) • Inoperable doors or windows within the wall line
Stucco	• Loose weep screed or gap between weep screed and sill plate (indicating detachment of stucco from studs) • Evidence of in-plane wall racking without significant stucco cracking (indicating detachment of stucco from studs)
Plaster on wood or gypsum lath	• Evidence of in-plane wall racking without significant plaster cracking (indicating detachment of plaster from studs)
Gypsum wallboard or plaster on gypsum lath	• Along panel edges, cracks in finished surface or tearing, buckling, or separation of joint tape (indicating panel rocking or sliding and related gouging of slots at fasteners in the back face of the panel) • Powder debris at the base of the wall (indicating gouging of the panel at fasteners)
Any wood sheathing type	• Extensive damage to architectural finishes (e.g., wallpaper, paneling, veneer) or exposed siding material (e.g., stucco, plaster, gypsum wallboard) • Inoperable doors or windows within the wall line • Loosened fasteners or fastener heads visible below paint or other finishes (indicating panel rocking)
Framing	• Evidence of damaging transient in-plane racking or out-of-plane leaning
Tie-downs	• Evidence of damaging uplift or wall rotation
Other load path elements	• Damage to adjacent architectural finishes consistent with relative movement between framing members
Gravity-load-carrying elements	• Deflection under load, or permanent sag, of supported floor or roof framing • Damage to architectural finishes consistent with relative movement between element in question and supported framing

Investigation Checklist 3-4 Wood-Frame Walls

Check if Present	*Element Type*	*Earthquake Damage*			*Non-EQ Damage*	*Not Investigated*
		Yes	*Suspected*	*No*		
☐	Horizontal wood siding *Repair threshold reminders:* • *Gap between siding and framing 1/4 inch* • *Split stud, loose nails, or torn building paper*	☐	☐	☐	☐	☐
☐	Plywood panel siding *Repair threshold reminders:* • *Gap between siding and framing* • *Panel rotation 1/4 inch from vertical* • *Nail tear through siding edge* • *Split stud or torn building paper*	☐	☐	☐	☐	☐

Investigation Checklist 3-4 Wood-Frame Walls (continued)

Check if Present	Element type	Earthquake Damage: Yes	Earthquake Damage: Suspected	Earthquake Damage: No	Non-EQ Damage	Not Investigated
☐	Stucco *Repair threshold reminders:* • *Crack 1/64 inch, 1/8 inch* • *Crack offset 1/16 inch* • *Extensive crack pattern* • *Stucco spall* • *Wire fracture* • *Separation from sheathing or framing 1/8 inch*	☐	☐	☐	☐	☐
☐	Plaster on wood lath *Repair threshold reminders:* • *Crack 1/32 inch, 1/8 inch* • *Extensive crack pattern*	☐	☐	☐	☐	☐
☐	Plaster on gypsum lath *Repair threshold reminders:* • *Crack 1/64 inch, 1/8 inch* • *Shifting and detachment of lath*	☐	☐	☐	☐	☐
☐	Gypsum wallboard *Repair threshold reminders:* • *Crack 1/64 inch, 1/8 inch* • *Nail pop* • *Fastener tear-out* • *Slotting of back face at fasteners 1/2 inch* • *Panel visibly rotated in-plane* • *Detachment from framing*	☐	☐	☐	☐	☐
☐	Wood structural panel [1], particleboard, or fiberboard sheathing ☐ Assumed ☐ Confirmed *Repair threshold reminders:* • *Nail tear through panel edge* • *Gap between panel and framing* • *Split framing*	☐	☐	☐	☐	☐
☐	Diagonal lumber sheathing ☐ Assumed ☐ Confirmed *Repair threshold reminders:* • *Split boards at nails, limited v. extensive* • *Fractured framing at nails*	☐	☐	☐	☐	☐

Investigation Checklist 3-4 Wood-Frame Walls (continued)

Check if Present	Element type	Earthquake Damage: Yes	Earthquake Damage: Suspected	Earthquake Damage: No	Non-EQ Damage	Not Investigated
☐	Horizontal lumber sheathing ☐ Assumed ☐ Confirmed *Repair threshold reminders:* • *Split boards at nails, local v. extensive* • *Fractured boards, extensive* • *Gap 1/4 inch between board and framing* • *Split stud, loose nails*	☐	☐	☐	☐	☐
☐	Framing [2] *Repair threshold reminders:* • *Racking or leaning 1/2 inch, 2 inch in story* • *Slip or gap between elements (e.g., studs, plates, floor framing) 1/4 inch* • *Splitting, local v. extensive* • *Crushing 1/8 inch*	☐	☐	☐	☐	☐
☐	Tie-downs [3] ☐ Assumed ☐ Confirmed *Repair threshold reminders:* • *Splitting of post or stud, local v. extensive* • *Damage to fasteners* • *Spalling of concrete at tie-down*	☐	☐	☐	☐	☐
☐	Other load path elements [4] ☐ Assumed ☐ Confirmed *Repair threshold reminders:* • *Nail withdrawal 1/8 inch, 1/4 inch* • *Chord or collector splitting, local v. extensive*	☐	☐	☐	☐	☐
☐	Gravity-load-carrying elements *Repair threshold reminders:* • *Post lean 1/4 inch per foot of height* • *Relative shift 1/4 inch between post and supported member*	☐	☐	☐	☐	☐

(1) Wood structural panel includes plywood and oriented strand board.
(2) Includes wood studs, headers, blocking, and other wall framing members.
(3) Includes tie-down anchors, rods, and fasteners.
(4) Includes clips, straps, and other metal hardware.

3.6 Other Seismic-Force-Resisting Elements

3.6.1 Scope

This section covers common seismic-force-resisting elements other than those involving conventional wood-frame walls.

- IRC Table R602.10.4 (or IBC Table 2308.6.3(1)) uses different designations. The element types below may be applied to these IRC bracing types as follows:
  - The portal frame type may be used for IRC portal frame at garage (PFG) and portal frame with hold-downs (PFH).
- Proprietary shear walls include pre-fabricated steel or wood elements. IRC alternate braced wall panels (ABW), which are similar but site-built, are covered in Section 3.5. See also the discussion of narrow wall pier and living space over garage configuration issues at Sections 2.2.4 and 2.2.5.
- The diagonal braces type covers elements used typically at the downhill end of a hillside house. The columns or posts may be wood or steel. The diagonal bracing may be lumber or steel rods or cables. Posts without diagonal bracing are covered as gravity-load-carrying elements in Section 3.5. See also the discussion of hillside house configuration issues at Section 2.2.3.
- The steel moment frame or cantilevered column type covers any such systems, proprietary or generic, pre-fabricated or site-built.

As with concealed wall sheathing, most of the seismic-force-resisting element types covered in this section are not directly observable. Therefore, the element type checklist includes check boxes to indicate whether the element type considered was actually confirmed or only assumed.

3.6.2 Indicators and Checklist

Table 3-5 Indicators of Damage to Concealed Other Seismic-Force-Resisting Elements

Element Being investigated	*Indicator*
Steel elements	• Damage to adjacent finish materials
Hillside houses: diagonal braces	• Leaning or racking of wood-frame side walls of the underfloor area • Evidence of floor diaphragm rotation or racking under cross-slope loading

Investigation Checklist 3-5 Other Seismic-Force-Resisting Elements

Check if Present	Element Type	Earthquake Damage: Yes	Earthquake Damage: Suspected	Earthquake Damage: No	Non-EQ Damage	Not Investigated
☐	Portal frames ☐ Assumed ☐ Confirmed *Repair threshold reminders:* • *Splitting of beam at beam-pier connection, limited v. extensive* • *Racking or leaning 1/2 inch in story, 2 inches in story*	☐	☐	☐	☐	☐
☐	Proprietary shear walls ☐ Assumed ☐ Confirmed *Repair threshold reminders:* • *Racking or leaning 1/2 inch in story, 2 inches in story*	☐	☐	☐	☐	☐
☐	Hillside houses: diagonal braces *Repair threshold reminders:* • *Fracture of wood members* *Fracture of rod braces or turnbuckles*	☐	☐	☐	☐	☐
☐	Steel moment frames or cantilevered columns ☐ Assumed ☐ Confirmed *Repair threshold reminders:* • *Racking or leaning 1/2 inch in 8-feet height, 2 inches in 8-feet height* • *Fracture of anchor bolts* • *Fracture at moment connection* • *Fracture at cantilevered column base*	☐	☐	☐	☐	☐

3.7 Floors, Ceilings, and Roofs

3.7.1 Scope

This section covers horizontal framing both as gravity-load-carrying elements and as diaphragms carrying lateral inertial loads.

- Floor, ceiling, or roof framing includes repetitive framing (joists), individual members (girders), and roof trusses, either pre-fabricated or site-built with conventional rafters.
- Rim joists are covered with framing in Section 3.5 because they typically form part of the load path between stories for perimeter wall systems.
- Loss of bearing on supporting posts or headers is covered in Section 3.5 with the framing or gravity-load-carrying element types.
- Diaphragms can include any sheathing or finish material.

3.7.2 Indicators and Checklist

Table 3-6 Indicators of Damage to Concealed Floors, Ceilings, and Roofs

Element Being Investigated	*Indicator*
Floor, ceiling, or roof framing	• Excessive deflection under live load, or permanent sag • Damage to walls, partitions, or other gravity-load-carrying elements that provide support for the framing • Evidence of roof leak (indicating damage to roofing or roof flashing related to roof distortion)
Floor, ceiling, or roof diaphragms	• Damage to wall or SFRS elements in the story below, in a pattern consistent with story torsion • Evidence of roof leak (indicating damage to roofing or roof flashing related to roof distortion) • Damage to exterior finishes at walls adjacent to a re-entrant corner • Distortion of pliable floor finishes (e.g., carpet, vinyl), especially adjacent to a reentrant corner • At gypsum board ceilings, tearing, buckling, or separation of tape joints along panel edges • At hillside houses, damage to the framing-to-foundation connection at the uphill end • At hillside houses, damage to the wood-frame walls or diagonally braced systems in the underfloor area

Investigation Checklist 3-6 Floors, Ceilings, and Roofs

Check if Present	*Element Type*	*Earthquake Damage*			*Non-EQ Damage*	*Not Investigated*
		Yes	*Suspected*	*No*		
☐	Floor, ceiling, or roof framing *Repair threshold reminders:* • *Loss of bearing 1/4 inch, 1 1/2 inches* • *Out-of-plane distortion 1/2 inch in 8 feet, 1 1/2 inches in 8 feet*	☐	☐	☐	☐	☐
☐	Floor diaphragms *Repair threshold reminders:* • *In-plane racking 1/2 inch in 8 feet, 2 inches in 8 feet*	☐	☐	☐	☐	☐
☐	Ceiling diaphragms *Repair threshold reminders:* • *Gypsum board crack 1/64 inch, 1/8 inch* • *Plaster crack 1/64 inch, 1/8 inch* • *Plaster cracking or spalling, local v. extensive* • *In-plane racking 1/2 inch in 8 feet, 2 inches in 8 feet* • *Slotting of back face at fasteners 1/2 inch*	☐	☐	☐	☐	☐

Investigation Checklist 3-6 Floors, Ceilings, and Roofs (continued)

Check if Present	Element Type	Earthquake Damage: Yes	Suspected	No	Non-EQ Damage	Not Investigated
☐	Roof diaphragms *Repair threshold reminders:* • *In-plane racking 1/2 inch in 8 feet, 2 inches in 8 feet* • *Fracture of lumber sheathing*	☐	☐	☐	☐	☐

3.8 Fireplaces and Chimneys

3.8.1 Scope

Fireplaces and chimneys are not normally considered structural elements, but the *Engineering Guidelines* cover them because interaction with and connection to the wood structural framing is a frequent source of earthquake damage.

- Only masonry chimneys and fireplaces are considered.
    - Precast concrete chimneys are not considered as elements themselves, but the checklist items for bracing and anchorage might apply, at the discretion of the structural consultant.
    - Light-frame chimneys are considered nonstructural elements.
- Both reinforced and unreinforced masonry elements are considered. While damage patterns can differ, repair techniques are similar, so there is no need to confirm the presence or adequacy of existing reinforcing during the initial investigation.
- Anchorage to house framing includes the wood floor, ceiling, or roof framing to which the anchorage is connected.
- Bracing to roof framing includes the wood roof framing to which the bracing is connected.
- Functionality and fire safety of the fireplace and chimney are outside the scope of this document. Potential damage to flue liners, dampers, and other elements related to the function of the fireplace are covered in the *General Guidelines*.

3.8.2 Indicators and Checklist

Table 3-7 Indicators of Damage to Concealed Fireplaces and Chimneys

Element Being Investigated	*Indicator*
Chimney or firebox	• Damage to adjacent finishes or enclosing walls
Anchorage to house framing	• Permanent chimney lean • Evidence of transient relative movement between chimney and house wall
Bracing to roof framing	• Damage to roofing adjacent to brace connection

Investigation Checklist 3-7 Fireplaces and Chimneys

Check if Present	Element Type	Earthquake Damage: Yes	Earthquake Damage: Suspected	Earthquake Damage: No	Non-EQ Damage	Not Investigated
☐	Chimneys or fireboxes *Repair threshold reminders:* • *Lean 1/2 inch over 8-feet height, 2 inches over 8-feet height* • *Crack in brick 1/16 inch, 1/8 inch* • *Crack in mortar 1/8 inch*	☐	☐	☐	☐	☐
☐	Anchorage to house framing	☐	☐	☐	☐	☐
☐	Bracing to roof framing	☐	☐	☐	☐	☐
☐	Interior fireplace surrounds *Repair threshold reminders:* • *Veneer detached 1/2 inch*	☐	☐	☐	☐	☐

3.9 Nonstructural Components

3.9.1 Scope

Nonstructural components are not structural elements, but this document covers the interaction of their anchorage or bracing with the wood structural framing, which is a frequent source of earthquake damage. The scope of this section includes components most likely to pose falling or fire hazards as a result of earthquake damage, and the damage investigation should focus on the soundness of the connection or anchorage to the supporting structure. The generic component types in the checklist are offered as broad reminders to the structural consultant.

- The emphasis of this section is on the performance of bracing and anchorage. Damage to the component itself is addressed by the *General Guidelines*.
- This section considers the bracing or anchorage of components attached to, but not completely built into, the house's wood framing. Window frames, door frames, and skylights, for example, are not considered (though damaged windows or doors can be indicators of damage to wood-frame walls; see Section 3.5).
- Mechanical, electrical, or plumbing (MEP) components include all systems typical for houses and braced or anchored to the wood framing or to wall, floor, or ceiling finishes. Typical MEP components include heating, ventilation, and air conditioning (HVAC) components, water heater tanks, solar panels, window-mounted air conditioners, roof- or wall-mounted satellite dishes, tall vent pipes or metal chimneys, ceiling fans, and chandeliers.
- Masonry chimneys and fireplaces are covered in Section 3.8.
- Architectural components include brick or masonry veneer (interior or exterior), tile or glass veneer, wall-mounted cabinets or casework, and ceramic or concrete roof tiles on pitched roofs or parapets. Other architectural components are covered elsewhere: stair framing in Section 3.5, ceilings in Section 3.7, chimney surrounds in Section 3.8, appurtenances in Section 3.10. Glazing in windows,

doors, or skylights is covered in the *General Guidelines*, but damage to these components can indicate damage to wood-frame walls (see Section 3.5).

- Related items outside the scope of the *Engineering Guidelines* include:
  - Functionality, fire safety, and potential hazardous material release from any equipment.
  - Damage unrelated to the performance of bracing or anchorage, including pipe leaks, roofing damage, or loss of weather protection.

3.9.2 Indicators and Checklist

Table 3-8 Indicators of Damage to Concealed Nonstructural Components

Component Being Investigated	*Indicator*
Mechanical or plumbing component bracing and anchorage	• For components in attics or within a ceiling space, damage to the ceiling below • For mechanical or plumbing components, evidence of leaks visible through ceiling or wall finishes below
Architectural component bracing and anchorage	• For exterior veneer, evidence of condensation, ice build-up, or leaks consistent with detached veneer

Investigation Checklist 3-8 Nonstructural Components

Check if Present	*Component Type*	*Earthquake Damage*			*Non-EQ Damage*	*Not Investigated*
		Yes	*Suspected*	*No*		
☐	Mechanical, electrical, or plumbing component bracing and anchorage	☐	☐	☐	☐	☐
☐	Architectural component bracing and anchorage	☐	☐	☐	☐	☐

3.10 Appurtenances

3.10.1 Scope

For the purposes of the *Engineering Guidelines*, appurtenances include only substructures attached to the house's main structure, usually with a distinct structural system. The emphasis of this section is on appurtenances that rely on the house's main structure for gravity support or for lateral resistance to ground shaking. The damage investigation should focus on the elements that provide support for the appurtenance itself (such as posts and wood framing) and on the soundness of the connection to the main structure (which is often made with a bolted ledger). The generic element types in the checklist are offered as broad reminders to the structural consultant.

- Common appurtenances within the scope of this section include covered porches or entryways, attached decks, attached carports or patio covers, and exterior stairs.

- Appurtenances built with typical house-like construction may use applicable element types from other sections in this chapter. Horizontal additions with no seismic-force-resisting systems of their own (such as an all-glass sunroom or screened porch) may use either this section or other applicable sections; damage investigation should focus on the connection of the addition to the main house. Vertical additions should be considered part of the main house, with additional attention to the interface between construction types.
- Detached occupiable structures (such as a garage, pool house, or gazebo), if within the scope of the investigation, should be considered full structures of their own and investigated separately.

3.10.2 Indicators and Checklist

For appurtenances built with typical house-like construction, the indicators applicable to other elements might apply.

Table 3-9 Indicators of Damage to Concealed Elements of Appurtenances

Element Being Investigated	*Indicator*
Connections of appurtenances to house	• Damage to the house's structural element providing gravity or lateral support for the appurtenance

Investigation Checklist 3-9 Appurtenances

Check if Present	*Element Type*	*Earthquake Damage*			*Non-EQ Damage*	*Not Investigated*
		Yes	*Suspected*	*No*		
☐	Appurtenances	☐	☐	☐	☐	☐
☐	Connections of appurtenances to house	☐	☐	☐	☐	☐

CHAPTER 4

Damage Repair: Structural

4.1 Introduction

This chapter addresses repair of earthquake damage that is determined to be structurally significant, as defined in Section 1.2. It is intended that the structural consultant use the procedures of this chapter to identify the methods and extent of repairs. This chapter is to be used following the completion of a damage investigation in accordance with Chapter 3.

This chapter is organized around elements of the house, where for each element, guidance on repair is presented. This organization is intended to allow users to read Section 4.1 and then proceed to sections addressing the specific elements identified during the damage investigation.

The damage investigation process described in Chapter 3 includes an initial visual and tactile investigation. Where indicators of damage to concealed elements are identified, a destructive investigation may subsequently occur. The structural consultant should use this chapter following each damage investigation. Where no damage is identified, no repair is needed. Where suspected damage (i.e., indicators of damage to concealed elements are present) is identified, see Section 3.1.2 for the intended treatment. If additional damage is identified during repairs to the house, this chapter can be used to identify repair methods for the additional damage.

For each element, Section 4.3 through Section 4.10 provide a series of damage pattern descriptions in tabular form, along with associated repair recommendations. The structural consultant should select the damage pattern most representative of the conditions observed during the damage investigation. Where different repairs can reasonably be applied to different portions of the element, the most representative repairs for each portion should be selected. Where observed damage patterns are less significant than those listed, repair is generally not warranted.

In some instances, the structural consultant will make the determination that the observed earthquake damage to some or all elements is not structurally significant; for such elements the required repairs should be determined in accordance with the *General Guidelines*. For these elements, it is anticipated that the structural consultant will transfer determination of required repairs back to the person who performed the general assessment. When this occurs, use of Chapter 4 will only address method and extent of repair for damage determined by the structural consultant to be structurally significant. Despite the intended division of scope between the *General Guidelines* and *Engineering Guidelines*, descriptions of damage and associated repairs in this chapter extend from minor to extensive, repeating and expanding upon the damage descriptions provided in the *General Guidelines*. This is done to aid users of the *Engineering Guidelines* in understanding the full range of possible damage patterns and repair methods.

Damage pattern descriptions and repair recommendations that are repeated from the *General Guidelines* are indicated in the repair tables with gray shading.

4.1.1 Repair Objective

Repair recommendations in this chapter are intended to provide structural performance of the house's gravity-load-carrying and seismic-force-resisting systems that is substantially equivalent to that of the pre-earthquake condition in terms of the expected damage states and related safety during future earthquakes as follows:

- For damage states or patterns associated with loss of peak strength, repairs are intended to substantially restore peak strength.
- For damage states or patterns associated with loss of stiffness judged to affect future performance, repairs are intended to substantially restore stiffness.

The consideration of loss of strength and stiffness incorporated into the repair recommendations of this chapter are derived from available research data and the experience and judgement of the developers of this document. Applicable research data is summarized in Appendix D. An element is considered to have lost peak strength when it has been loaded to or beyond its peak strength. Otherwise, the peak strength is judged to not have been reduced, and structural safety performance in future earthquakes is expected to be substantially equivalent. This can be seen in a wide variety of cyclic tests of seismic-force-resisting elements. However, if an element has been loaded to or beyond its peak strength, the repairs recommended in this document are intended to substantially restore peak strength.

Compared to loss of strength, determining if loss of stiffness will impact performance in future earthquakes requires more engineering judgment. Hence, the repair recommendations rely more heavily on the judgement of the developers of this document. Loss of stiffness at low levels of loading or without visible signs of significant damage is judged not to have meaningfully compromised an element with respect to its structural performance in future earthquakes. However, if an element has lost stiffness judged to affect future performance, the repairs recommended in this document are intended to substantially restore stiffness.

What has been described above is often referred to as "in-kind" or "like-kind" repair. This repair objective is differentiated from repairs that include upgrades, as discussed in Section 4.1.4. In some cases, the in-kind repair of older construction materials is not practical, in which case an equivalent in modern materials is recommended in this document.

Because repairs are based on in-kind replacement of elements, specific consideration of the ductility of repair materials and methods is not required. For repair by other than in-kind replacement, equivalent ductility might be an appropriate consideration, in addition to consideration of strength and stiffness.

4.1.2 Determination of Repair Method and Extent

The repair method and extent are to be determined by the structural consultant using the procedures of this chapter. This chapter provides a description of typical repair methods for each element for common earthquake damage patterns. This is done with the understanding that other elements, patterns, or considerations are possible. The structural consultant is encouraged to first systematically consider the damage patterns discussed in this chapter. Only consider other, unlisted damage patterns if there is compelling rationale to do so.

The appropriate repair of earthquake damage must consider the nature, extent, and significance of the damage. The end determination of best method of repair should consider the cumulative impact of damage to each element. In some cases, repair of a significant amount of moderate damage may be more expensive than removal and replacement of materials. When this is true, replacement of materials may be the appropriate approach.

The extent of repairs is intended to be determined based on the nature, extent, and significance of damage documented during the damage investigation. The extent of repairs should be determined for each of the repair methods selected. However, other factors unrelated to structural integrity, such as maintaining a reasonable level of visual uniformity, can be appropriate and are discussed throughout this chapter and the *General Guidelines*.

Repairs are to be developed for earthquake damage and pre-earthquake damage that is believed to have been worsened by the earthquake, such that the earthquake has made the damage categorically different. See Section 3.1.2 for additional information.

The repair guidance in this chapter is intended to be used to develop the repair section of the technical consultant report, as discussed in Chapter 8. Implementation of repairs will sometimes require development of construction documents providing details of repair. The development of construction documents, however, is beyond the scope of this document.

4.1.3 Hazardous Materials

The repair methods presented in this chapter presume that building materials are free of regulated levels of hazardous materials. If hazardous materials, including regulated levels of lead or asbestos, are potentially present, the abatement and waste disposal recommendations of an environmental consultant should be incorporated into the overall scope of repair.

4.1.4 Repair versus Upgrade

As discussed in Section 1.4.1, this chapter addresses repairs required to return the house to its pre-earthquake state. The locally adopted version of the building or residential code may require upgrade of all or portions of the house construction in conjunction with the repairs. Such code-triggered upgrades are beyond the scope of this chapter but are discussed further in Appendix A.

As part of damage repair, it may be desirable to incorporate voluntary upgrade measures. This is particularly true where the structural consultant identifies a common earthquake vulnerability (such as one of the vulnerabilities discussed in Section 2.2) while conducting the damage investigation. Implementation of voluntary upgrades are broadly permitted by building codes, but the structural consultant should consult with the local building department to identify any applicable local requirements. Voluntary upgrades are beyond the scope of this chapter but are discussed further in Appendix B.

4.1.5 Historic Structures

Where the guidance in this section is being applied to eligible or designated historic structures, different criteria for repair (such as the criteria provided in applicable historical building code provisions) may be appropriate in order to preserve historic fabric. This is true of several aspects of repair; in particular, it may be appropriate to stabilize the house at a larger-than-typical distortion or to retain finish materials that might otherwise be replaced.

4.2 Ground

Earthquake-induced permanent ground deformation is covered in Chapter 5 through Chapter 7. Where the need is identified by the structural consultant, repair recommendations for foundations, slabs-on-grade, and basement walls (Section 4.3) should be determined collaboratively by the structural and geotechnical consultants. Section 3.2 provides guidance to the structural consultant about indicators of earthquake-induced permanent ground deformation and when the use of a geotechnical consultant should be considered.

4.3 Foundations, Slabs-on-Grade, and Basement Walls

This section addresses repair of slab-on-grade floors and associated footings, concrete and masonry footings, concrete and masonry stemwalls and basement walls, and post and pier foundation systems. Section 4.3.1 through Section 4.3.6 provide repair recommendations for damage patterns associated with each foundation type. Section 4.3.7 provides additional discussion of damage patterns common in these foundation systems. Section 4.3.8 provides additional discussion of common repair methods.

Terms used in the Section 4.3 tables to describe damage patterns for foundations, slabs-on-grade, and basement walls can be found in the Glossary (see “foundation damage patterns”). These terms include settlement, spreading, offset, slope, and loss of bearing.

4.3.1 Slabs-on-Grade

Slabs-on-grade involve concrete slabs intended to be cast on and continuously supported by underlying soils or subgrade. Slabs that are not continuously supported are beyond the scope of this section. Addressed in this section are slabs-on-grade cast independently from concrete footings (common in older garages and some houses), slabs-on-grade cast on top of continuous concrete footings, and slabs-on-grade cast integrally with continuous concrete footings.

Repairs to slabs-on-grade and associated footings are to be determined based on the earthquake damage patterns provided in Table 4-1. Earthquake damage patterns and repairs provided in Section 4.3.1 are primarily intended for slabs that are covered by finish materials (e.g., carpet, tile, wood flooring) or that are located in areas where uncovered repairs are aesthetically acceptable (e.g., garages, basements). See additional discussion in Section 4.3.8 for slabs that are exposed to view during normal use.

Table 4-1 Repair of Slabs-on-Grade and Associated Footings [1, 2]

Earthquake Damage Pattern [3]		*Repair Method* [4]
4-1A	Damage to stucco finish along horizontal cold joint between bottom of slab and top of footing, no apparent footing damage, no evidence of sliding more than 1/8 inch	No structural repair of concrete. See Section 4.5.3 for stucco repair
4-1B	Opening of construction joint between edge of slab and adjacent wall footing caused by the earthquake (typical garage detail)	Seal gap with epoxy or grout
4-1C	Crack in slab or footing Width: less than 1/8 inch Settlement, spreading, offset, or slope: slight to none	Seal cracks with epoxy if there is potential for water or pest intrusion, otherwise no repair
4-1D	Crack in slab or footing Width: up to 1/2 inch Offset: up to 1/16 inch Settlement, spreading or slope: slight to none	Seal crack with epoxy or epoxy and aggregate
4-1E	Local spall in concrete	Patch with repair mortar
4-1F	Crack in slab Width: more than 1/2 inch Settlement, spreading, offset or slope: slight to none	Seal visible cracks with epoxy or grout
4-1G	Crack in slab Width: more than 1/2 inch Settlement, spreading, offset or slope: over limited area internal to the slab	Seal visible cracks with epoxy or grout, apply leveling compound
4-1H	Crack in footing Width: more than 1/2 inch Settlement, spreading, offset or slope: observable	Remove and replace several feet of footing length [5]
4-1I	Vertical offset or slope in slab or footing Change in elevation: 1/2 inch or more in 8 feet	Epoxy seal any visible cracks, apply leveling compound [5]
4-1J	Vertical offset or slope in slab or footing Change in elevation: 2 inches or more in 8 feet	Stabilize soil if required. Repair or reconstruct the slab-on-grade and associated footing [5]
4-1K	Loss of bearing (soil settled away from slab or footing)	Cast new grout or lean concrete under existing slab-on-grade and associated footing [5]

[1] Damage patterns with gray shading are also identified in the *General Guidelines*.

[2] Identify damage pattern and repair where damage has been identified during the damage investigation. Select damage pattern most representative of observed damage caused by or worsened by the earthquake. See Section 3.1.2 and Section 4.1.2 for discussion of worsened damage.

[3] See Section 4.3.7 for discussion of earthquake damage patterns.

[4] See Section 4.3.8 for discussion of repair methods.

[5] Involvement of a geotechnical consultant should be considered. See Section 4.2.

4.3.2 Footings and Stemwalls, Concrete

Concrete footings can be found in combination with both concrete and masonry stemwalls. Concrete stemwalls are addressed in this section. Masonry stemwalls are addressed in Section 4.3.3. Foundation repairs are to be determined based on the earthquake damage patterns provided in Table 4-2. More detailed discussion of damage patterns and repair methods is provided in Section 4.3.7 and Section 4.3.8.

Table 4-2 Repair of Footings and Stemwalls, Concrete [1, 2]

	Earthquake Damage Pattern [3]	*Repair Method* [4]
4-2A	Crack in footing or stemwall Width: less than 1/8 inch Settlement, spreading, offset or slope: slight to none	Seal cracks if there is potential for water or pest intrusion, otherwise no repair
4-2B	Crack in footing or stemwall Width: up to 1/2 inch Offset: up to 1/16 inch Settlement, spreading, or slope: slight to none	Seal crack with epoxy or epoxy and aggregate
4-2C	Local spall in concrete	Patch with repair mortar
4-2D	Crack in footing or stemwall Width: more than 1/2 inch Settlement or slope: slight to none Spreading or offset: not more than 1 inch	Remove and replace several feet of footing and stemwall length or augment footing and stemwall from crawlspace [5]
4-2E	Vertical offset or slope in footing Change in elevation of 1/2 inch or more in 8 feet	Epoxy seal any visible cracks in footing and stemwall. Take practical measures to relevel the floor framing above including shimming between top of footing or stemwall and floor framing or place leveling compound under floor finish[5]
4-2F	Vertical offset or slope in footing Total change in elevation of 2 inches or more in 8 feet	Stabilize foundation where required. Relevel the floor framing and superstructure by either shimming between top of footing or stemwall and floor framing, or where required, underpinning or reconstructing the footing and stemwall [5]
4-2G	Leaning of stemwall More than 1/4 inch per foot of wall height	Stabilize foundation where required. Relevel the floor framing and superstructure by either shimming between top of footing or stemwall and floor framing, or where required, underpinning or reconstructing the footing and stemwall [5]
4-2H	Loss of bearing (soil settled away from footing)	Cast new grout or lean concrete under existing footing and stemwall [5]

(1) Damage patterns with gray shading are also identified in the *General Guidelines*.

(2) Identify damage pattern and repair where damage has been identified during the damage investigation. Select damage pattern most representative of observed damage caused by or worsened by the earthquake. See Section 3.1.2 and Section 4.1.2 for discussion of worsened damage.

(3) See Section 4.3.7 for discussion of earthquake damage patterns.

(4) See Section 4.3.8 for discussion of repair methods.

(5) Involvement of a geotechnical consultant should be considered. See Section 4.2.

4.3.3 Footings and Stemwalls, Masonry

Masonry footings in houses are most commonly unreinforced brick footings or combined footings and stemwalls. While cut stone and rubble stone masonry footings occur in houses, these are not commonly occurring and are not addressed by this document. Concrete masonry stemwalls are also commonly found in combination with concrete footings. Concrete masonry stemwalls found in house construction can include ungrouted, partially grouted, and fully grouted construction, and can be unreinforced or reinforced. In particular, the presence or lack of grout can have an impact on repair methods.

Repairs to brick masonry footings or combined footings and stemwalls are to be determined based on the earthquake damage patterns provided in Table 4-3. Repairs to concrete masonry stemwalls are to be determined from the damage patterns in Table 4-4. More detailed discussion of damage patterns and repair methods is provided in Sections 4.3.7 and 4.3.8.

Table 4-3 Repair of Footings and Stemwalls, Brick Masonry [1,2]

Earthquake Damage Pattern [3]		*Repair Method [4]*
4-3A	Crack in brick or mortar joint Width: less than 1/8 inch Settlement, spreading, offset or slope: slight to none	Seal cracks if there is potential for water or pest intrusion, otherwise no repair
4-3B	Crack in brick or mortar joint Width: up to 1/2 inch Offset: up to 1/16 inch Settlement, spreading, or slope: slight to none	Repoint mortar joints. Remove and replace cracked bricks
4-3C	Local spall in masonry	Patch with repair mortar or remove and replace spalled bricks
4-3D	Crack in footing or stemwall Width: more than 1/2 inch Settlement, spreading, or slope: observable	Remove and reconstruct or augment damaged portion of the foundation [5]
4-3E	Vertical drop or slope in footing or stemwall Change in elevation of 1/2 inch or more in 8 feet	Repair with epoxy or mortar any visible cracks in footing and stemwall. Take practical measures to relevel the floor framing above, including shimming between top of footing or stemwall and floor framing, or place leveling compound under floor finish [5]
4-3F	Vertical drop or slope in footing or stemwall Change in elevation of 2 inches or more in 8 feet	Stabilize foundation where required. Relevel the floor framing and superstructure by either shimming between top of footing or stemwall and floor framing, or where required, underpinning or reconstructing the footing and stemwall [5]
4-3G	Leaning of stemwall More than 1/4 inch per foot of wall height	Stabilize foundation where required. Relevel the floor framing and superstructure by either shimming between top of footing or stemwall and floor framing, or where required, underpinning or reconstructing the footing and stemwall [5]
4-3H	Loss of bearing (soil settled away from footing)	Cast new concrete under existing foundation [5]

Table 4-3 Repair of Footings and Stemwalls, Brick Masonry [1, 2] (continued)

(1) Damage patterns with gray shading are also identified in the *General Guidelines*.

(2) Identify damage pattern and repair where damage has been identified during the damage investigation. Select damage pattern most representative of observed damage caused by or worsened by the earthquake. See Section 3.1.2 and Section 4.1.2 for discussion of worsened damage.

(3) See Section 4.3.7 for discussion of earthquake damage patterns.

(4) See Section 4.3.8 for discussion of repair methods.

(5) Involvement of a geotechnical consultant should be considered. See Section 4.2.

Table 4-4 Repair of Stemwalls, Concrete Masonry [1,2]

	Earthquake Damage Pattern [3]	*Repair Method [4]*
4-4A	Crack in masonry unit or mortar joint Width: less than 1/8 inch Settlement, spreading, offset, or slope: slight to none	Seal cracks if there is potential for water or pest intrusion, otherwise no repair
4-4B	Local spall in masonry	Patch with repair mortar or remove and replace spalled units
4-4C	Crack in mortar joint Width: up to 1/2 inch Offset: up to 1/16 inch Settlement, spreading, offset, or slope: slight to none	Repoint with mortar or seal with epoxy or epoxy and aggregate
4-4D	Crack in ungrouted masonry unit Width: up to 1/2 inch Offset: up to 1/16 inch Settlement, spreading, offset, or slope: slight to none	Remove and replace cracked units or augment stemwall
4-4E	Crack in grouted masonry unit Width: up to 1/2 inch Offset: up to 1/16 inch Settlement, spreading, offset, or slope: slight to none	Seal with epoxy or epoxy and aggregate
4-4F	Crack in grouted or ungrouted masonry Width: more than 1/2 inch Settlement, spreading, or slope: observable	Remove and reconstruct or augment damaged portion of the stemwall [5]
4-4G	Vertical drop or slope in stemwall Change in elevation of 1/2 inch or more in 8 feet	Epoxy or grout seal any visible cracks in the stemwall. Take practical measures to relevel the floor framing above, including shimming between top of stemwall and floor framing, or place leveling compound under floor finish [5]
4-4H	Vertical drop or slope in stemwall Change in elevation of 2 inches or more in 8 feet	Stabilize foundation where required. Relevel the floor framing and superstructure by either shimming between top of stemwall and floor framing, or where required, underpinning or reconstructing the footing and stemwall [5]
4-4I	Leaning of stemwall More than 1/4 inch per foot of wall height	Stabilize foundation where required. Relevel the floor framing and superstructure by either shimming between top of stemwall and floor framing, or where required, underpinning or reconstructing the footing and stemwall [5]

Table 4-4 Repair of Stemwalls, Concrete Masonry [1,2] (continued)

Earthquake Damage Pattern [3]		Repair Method [4]
4-4J	Loss of bearing (soil settled away from footing)	Cast new grout or lean concrete under existing foundations [5]

[1] Damage patterns with gray shading are also identified in the *General Guidelines*.

[2] Identify damage pattern and repair only where damage has been identified during the damage investigation. Select damage pattern most representative of observed damage caused by or worsened by the earthquake. See Section 3.1.2 and Section 4.1.2 for discussion of worsened damage.

[3] See Section 4.3.7 for discussion of earthquake damage patterns.

[4] See Section 4.3.8 for discussion of repair methods.

[5] Involvement of a geotechnical consultant should be considered. See Section 4.2.

4.3.4 Post and Pier Foundations

In post and pier foundations, the house is supported by wood posts extending between the lowest floor of the house and a series of isolated pier footings either at the house interior (in combination with a cripple wall or stemwall at the house perimeter) or at both the house perimeter and interior. The foundation piers can be shallow isolated concrete piers or deeper drilled piers or caissons.

Repairs to post and pier foundation systems are to be determined based on the earthquake damage patterns provided in Table 4-5. More detailed discussion of damage patterns and repair methods is provided in Section 4.3.7 and Section 4.3.8.

Table 4-5 Repair of Post and Pier Foundation Systems [1,2]

Earthquake Damage Pattern [3]		Repair Method [4]
4-5A	Post lean Up to 1/2 inch over the post height due to shifting of the house	No repair
4-5B	Post shifted on the footing due to shifting of the house Post is vertical Post falls in the center third of the footing horizontal dimension	Refasten post to the pier footing
4-5C	Post lean Up to 1/4 inch per foot of height due to shifting of the house	Take practical measures to return house to as close to original position as possible. Refasten post top and bottom and refasten any kickers bracing the post
4-5D	Post lean More than 1/4 inch per foot of height due to shifting of the house	Return house to as close to original position as possible. Reconstruct posts as required to return to original condition. Temporary shoring may be required
4-5E	Post shifted on the footing due to shifting of the house Post is vertical Post falls outside the center third of the footing horizontal dimension	Return house to as close to original position as possible. If post still falls outside center third of footing, augment or replace shallow pier footing, provide engineered repair for deep pier foundations

Table 4-5 Repair of Post and Pier Foundation Systems (1,2) (continued)

Earthquake Damage Pattern (3)		Repair Method (4)
4-5F	Pier block rolled away from original position	Replace with new cast-in-place pier (5)
4-5G	Vertical drop in floor supported by pier Change in elevation of 1/16 inch per foot and 1/4 inch total or more over 8 feet	Return house to as close to original elevation as possible. Shim and refasten post top and bottom and refasten any kickers bracing the post, replace post if needed (5)
4-5H	Vertical drop in floor supported by pier Change in elevation of 1/4 inch per foot and 2 inches total or more over 8 feet	Stabilize foundation where required. Replace post and repair or reconstruct the foundations if needed (5)
4-5I	Deep pier foundation is shifted more than 2 inches	Stabilize foundation where required. Provide engineered repair or reconstruct the foundations (5)
4-5J	Deep pier foundation has cracking or spalling damage	Provide engineered repair using repair grout and epoxy dowels

(1) Damage patterns with gray shading are also identified in the *General Guidelines*.

(2) Identify damage pattern and repair where damage has been identified during the damage investigation. Select damage pattern most representative of observed damage caused by or worsened by the earthquake. See Section 3.1.2 and Section 4.1.2 for discussion of worsened damage.

(3) See Section 4.3.7 for discussion of earthquake damage patterns

(4) See Section 4.3.8 for discussion of repair methods.

(5) Involvement of a geotechnical consultant should be considered. See Section 4.2.

4.3.5 Basement Walls, Concrete

Repairs to concrete basement walls are to be determined based on the earthquake damage patterns provided in Table 4-6. More detailed discussion of damage patterns and repair methods is provided in Section 4.3.7 and Section 4.3.8.

Table 4-6 Repair of Concrete Basement Walls (1,2)

Earthquake Damage Pattern (3)		Repair Method (4)
4-6A	Crack in basement wall Width: less than 1/8 inch Settlement, spreading, offset, or slope: slight to none	Seal cracks if there is potential for water or pest intrusion, otherwise no repair
4-6B	Local spall in concrete	Patch with repair mortar
4-6C	Crack in basement wall Width: less than 1/2 inch Offset: up to 1/16 inch Settlement, spreading, offset, or slope: slight to none	Seal with epoxy or epoxy and aggregate
4-6D	Crack in basement wall Width: more than 1/2 inch Settlement, spreading, offset or slope: observable	Where possible, repair basement wall from interior face by local patching or augmentation. Only where necessary, remove several feet of basement wall length, epoxy rebar dowels to remaining concrete on each side (size and spacing to match existing, but not less than No. 4 at 16 inches on center). Recast concrete. Maintain or reestablish drainage and waterproofing of backfill, where exists

Table 4-6 Repair of Concrete Basement Walls [1,2] (continued)

	Earthquake Damage Pattern [3]	*Repair Method [4]*
4-6E	Vertical drop or slope in basement wall Change in elevation of 1/2 inch or more in 8 feet	Epoxy seal any visible cracks in footing and basement wall. Take practical measures to relevel the floor framing above, including shimming between top of basement wall and floor framing, or place leveling compound under floor finish [5]
4-6F	Vertical drop or slope in basement wall Change in elevation of 2 inches or more in 8 feet	Stabilize basement wall and foundation where required. Where possible, relevel framed floor and superstructure by shimming between top of foundation and framing. Reconstruct or underpin foundation where required [5]
4-6G	Leaning of the basement wall over partial height or full height More than 1/4 inch per foot of wall height	Stabilize foundation where required. Relevel the floor framing and superstructure by either shimming between top of basement wall and floor framing, or where required, underpinning or reconstructing the footing and basement wall [5]
4-6H	Loss of bearing (soil settled away from footing)	Cast new grout or lean concrete under existing basement wall and foundation [5]

(1) Damage patterns with gray shading are also identified in the *General Guidelines*.

(2) Identify damage pattern and repair where damage has been identified during the damage investigation. Select damage pattern most representative of observed damage caused by or worsened by the earthquake. See Section 3.1.2 and Section 4.1.2 for discussion of worsened damage.

(3) See Section 4.3.7 for discussion of earthquake damage patterns.

(4) See Section 4.3.8 for discussion of repair methods.

(5) Involvement of a geotechnical consultant should be considered. See Section 4.2.

4.3.6 Basement Walls, Masonry

Repairs to masonry basement walls are to be determined based on the earthquake damage patterns provided in Table 4-7. More detailed discussion of damage patterns and repair methods is provided in Section 4.3.7 and Section 4.3.8.

Table 4-7 Repair of Masonry Basement Walls [1,2]

	Earthquake Damage Pattern [3]	*Repair Method [4]*
4-7A	Crack in basement wall Width: less than 1/8 inch Settlement, spreading, offset, or slope: slight to none	Seal cracks if there is potential for water or pest intrusion, otherwise no repair
4-7B	Local spall in masonry	Patch with repair mortar or remove and replace spalled units
4-7C	Crack in basement wall Width: less than 1/2 inch Offset: up to 1/16 inch Settlement, spreading, offset, or slope: slight to none	Repoint with mortar or seal with epoxy or epoxy and aggregate

Table 4-7 Repair of Masonry Basement Walls [1,2] (continued)

Earthquake Damage Pattern [3]		*Repair Method [4]*
4-7D	Crack in basement wall Width: more than 1/2 inch Settlement, spreading, offset or slope: observable	Where possible, repair basement wall from interior face by local patching or augmentation. Only where necessary remove several feet of basement wall length, epoxy rebar dowels to remaining masonry on each side (size and spacing to match existing, but not less than No. 4 at 16 inches on center). Cast concrete. Use masonry face shells if needed to maintain visual appearance. Maintain or reestablish drainage and waterproofing of backfill, where exists
4-7E	Vertical drop or slope in basement wall Change in elevation of 1/2 inch or more in 8 feet	Epoxy seal any visible cracks in basement wall. Take practical measures to relevel the floor framing above, including shimming between top of basement wall and floor framing, or placing leveling compound under floor finish [5]
4-7F	Vertical drop or slope in basement wall Change in elevation of 2 inches or more in 8 feet	Stabilize foundation where required. Where possible, relevel framed floor and superstructure by shimming between top of foundation and framing. Reconstruct foundation where required [5]
4-7G	Leaning of the basement wall More than 1/4 inch per foot of wall height	Stabilize foundation where required. Relevel the floor framing and superstructure by either shimming between top of basement wall and floor framing, or where required, underpinning or reconstructing the footing and basement wall [5]
4-7H	Loss of bearing (soil settled away from footing)	Cast new grout or lean concrete under existing basement wall and foundation [5]

(1) Damage patterns with gray shading are also identified in the *General Guidelines*.

(2) Identify damage pattern and repair where damage has been identified during the damage investigation. Select damage pattern most representative of observed damage caused by or worsened by the earthquake. See Section 3.1.2 and Section 4.1.2 for discussion of worsened damage.

(3) See Section 4.3.7 for discussion of earthquake damage patterns.

(4) See Section 4.3.8 for discussion of repair methods.

(5) Involvement of a geotechnical consultant should be considered. See Section 4.2.

4.3.7 Damage Patterns in Foundations, Slabs-on-Grade, and Basement Walls

For many cracks in residential concrete and masonry foundations, whether caused by an earthquake or not, the appropriate structural repair is to do nothing. These cracks are typically the result of shrinkage and have no practical effect on the function of the element. In the normal course of construction and maintenance of buildings, shrinkage cracks are ignored or cosmetically patched. Isolated cracks less than 1/2 inch with less than 1/16-inch out-of-plane offset are structurally insignificant and may be left unrepaired. Nonstructural repair of the crack may be appropriate for cosmetic reasons or to prevent moisture and pest intrusion.

Larger cracks or gaps in concrete and masonry foundations may provide an additional path of entry for insects, pests or moisture, and sealing of those cracks or cold joints may be desirable in some circumstances. Where no out-of-plane offset is present, cracks and gaps may be sealed by routing the crack if required and filling the space with an elastomeric joint sealant. Cracks and gaps may also be sealed with any number of commercially available cement-based patching materials, many of which have a latex additive to increase bond strength. For cosmetic considerations of exposed concrete, see Section 4.3.8.

The change in floor elevation of 1/2 inch in 8 feet (Figure 4-1) is intended to be measured over an 8-foot distance in order to capture large-scale movement, rather than very local conditions. This level of change in elevation occurs reasonably often in houses due to non-earthquake-related soil movement. Because such soil movement generally occurs over an extended period of time, there is often no associated damage to the house superstructure. This earthquake-related movement might be associated with damage to the house superstructure. The magnitude of the change in elevation has been selected to match damage patterns related to wood joist wall racking or leaning of 1/2 inch over a story height, discussed in Section 4.5.10. Where the 1/2-inch-in-8-feet is expressed as a lean, it is noted as 1/16 inch per foot.

The change in floor elevation of 2 inches in 8 feet (Figure 4-1) is intended to be measured over an 8-foot distance in order to capture large-scale movement, rather than very local conditions. Once a change in elevation of 2 inches in 8 feet has been reached, damage to the house superstructure is likely. This level of change in elevation could lead to structural stability concerns in the supported superstructure. Where the 2-inches-in-8 feet is expressed as a lean, it is noted as 1/4 inch per foot.

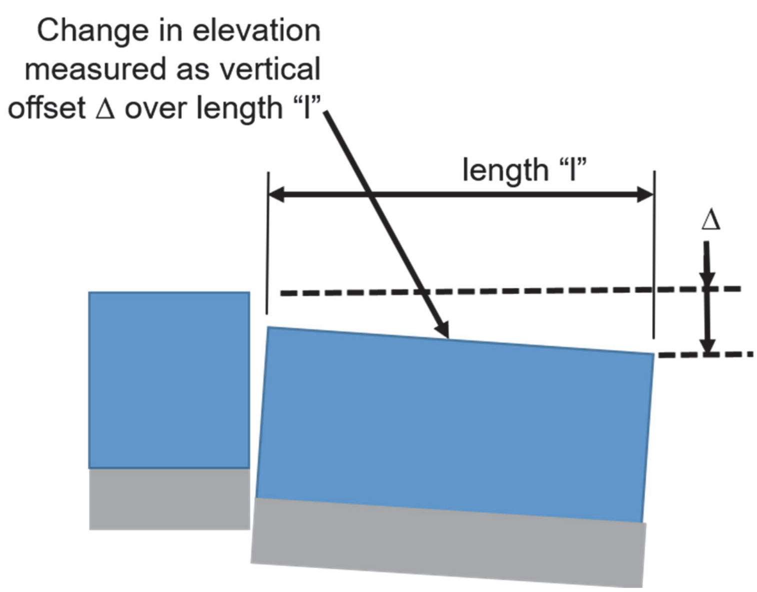


Figure 4-1 Measurement of change in elevation due to a vertical offset, slope, or both.

Following the September 2010 Darfield and 2011 Lyttelton, New Zealand earthquakes, the New Zealand Ministry of Business, Innovation, and Employment developed guidance for the assessment and repair of earthquake damaged houses. Included in this effort was development of damage descriptions and repair methods very similar in concept to those of Section 4.3 for houses that had experienced liquefaction of the supporting soils (MBIE, 2012). In addition to criteria related to the local change in floor elevation, as just discussed, the guidance included maximum changes in elevation over the entire house footprint; these were driven by judgement on the extent of releveling that is practical given the releveling methods being considered (e.g., shimming between floor framing and foundations, pressure grouting under slab-on-grade foundations). Similar overall limits might be applicable to the repairs discussed in this section where significant permanent ground deformation has occurred and should be considered in determining method and extent of repair.

4.3.8 Repair Methods for Foundations, Slabs-on-Grade, and Basement Walls

The primary intent of repair to slabs-on-grade is to return the slab to a condition that is functional as a floor surface. The primary intent of repairs to cracked foundations is to reconnect sections of foundation to regain continuity, strength, and stiffness. Where foundation differential settlement has occurred, the primary intent is to return the supported floors and superstructure to a structurally safe condition. Where loss of foundation bearing capacity has occurred, the primary intent of repairs is to regain competent bearing. The following repairs are believed to be common and practical methods of achieving these intents; other methods are possible and could potentially be more practical for a given repair situation.

Leveling compound. Where a vertical offset occurs across a crack in a floor slab, a leveling compound may be used to restore flatness as needed for installation of flooring. Floor covering installers routinely do minor leveling of slab surfaces to provide a smooth substrate below the floor finish. Where placement of leveling compound extends to full rooms or large portions of rooms, adjustment to baseboards and other trim and doors may also be required.

Epoxy injection. Epoxy (or adhesive) injection consists of the injection under controlled pressure, of epoxy resins or other adhesives formulated for structural repair of concrete, into cracks in concrete elements. Epoxy injection is a widely used method for concrete crack repair because it solves several problems: it seals the crack against water and pest entry, it protects any reinforcing steel crossing the crack from corrosion, and it provides tensile and flexural strength comparable to or exceeding that of the uncracked concrete element. Cracks ranging in width from 0.002 inches to 1/4 inch may be satisfactorily repaired with epoxy injection, as discussed in detail in Appendix G of the *General Guidelines*. For cosmetic considerations of exposed concrete, see Section 4.8.6 of the *General Guidelines*.

Seal cracks with epoxy. Cracks up to 1/4-inch wide can be sealed with any number of commercially available epoxy injection systems. Cracks up to approximately 1/2-inch wide can be sealed using a combination of epoxy and packed aggregate. See further discussion in the *General Guidelines*.

Seal cracks with grout. Cracks and gaps more than 1/4-inch wide can be sealed with any number of commercially available cement-based patching materials, many of which have a latex additive to increase bond strength.

Patch spalls with repair grout. Very small spalls can be prepared by chipping a beveled edge at the perimeter to key the patch material in. Patches for larger spalls should be held in place either with epoxied stainless steel pins, or with removal of existing concrete to 1 inch past existing reinforcing steel, such that the patch material will surround the reinforcing. Very large spalls might require added reinforcing steel. These methods will help avoid patches later popping off. There are a wide variety of repair grouts available. All should be used in accordance with the manufacturer recommendations for preparation and placement.

New grout or lean concrete under existing foundation. Where loss of bearing occurs due to permanent ground deformation, repair by casting new grout or lean concrete under the existing foundation, with adequate embedment into existing soils. This includes controlled low-strength material, such as self-consolidating concrete and similar.

Relevel floor and superstructure framing. Where the lowest floor is wood framed, it is often possible to relevel the floor and superstructure above by jacking and shimming between the top of the foundation, stemwall, or basement wall and the floor framing. Where this is done, the anchorage between the floor framing and the top of foundation, stemwall, or basement wall needs to be reestablished.

Augmentation of existing basement walls. Removal and replacement of portions of existing basement walls can be unwieldy. For this reason, augmentation of cracked or spalled basement walls from the interior face should be considered. This can involve the addition of repair mortars, concrete, or steel strengthening.

Removal and replacement of section of foundation. Remove and replace several feet of foundation length, provide epoxy-anchored rebar dowels top and bottom of remaining concrete on each side of the removed section, and recast concrete. Where removal and replacement is indicated, consideration should be given to the extent of removal and replacement. In general the extent should be as small as practical so as to not require partial reconstruction of other portions of the house (e.g., framing and finishes at bottom of walls). In some cases, it is most practical to include a slightly larger area of removal and replacement, for example to the full extent of a particular room or portion of a room. Where reconstruction would require significant shoring effort to support the house above, alternatives such as augmenting footings rather than replacing them should be considered.

Appearance of exposed concrete. As discussed in the *General Guidelines*, appearance is a consideration for some concrete repairs. Aesthetically acceptable repair of cracks in concrete exposed to view during regular use of the house (e.g., driveways, walkways, patios, and pool decks) is generally impractical, although there are methods available for that purpose. Generally, however, for architecturally exposed residential concrete slabs-on-grade (e.g., walks, patios), removal of the damaged area and replacement in-kind is usually the most practical solution.

Stabilization and repair. Where stabilization of the supporting soils is needed, the scope of the footing and slab repair work needs to be determined in conjunction with geotechnical consultant recommendations.

Repointing of brick masonry. It is generally possible to partially remove (scrape out) a mortar joint and replace it with new mortar (called repointing). Removing and replacing approximately one-half of the mortar joint depth will substantially restore pre-damage strength. Attention is needed to selecting joint mortar material that matches the existing mortar.

Removal and reconstruction or augmentation of brick masonry. Where a crack is more than 1/4-inch wide, the foundation can be repaired by removing and replacing the brick masonry in the vicinity of the crack. Where appearance is of concern, the bricks that are removed can be cleaned and reintegrated into the reconstruction. Shoring of the house may be required during reconstruction. Instead of repairing the existing masonry, it is sometimes appropriate to cast a new concrete foundation alongside and doweled into the masonry foundation. Where this approach is used, the new foundation should extend past the area of damage, possibly extending for the full length of the wall line.

Replace concrete masonry units (CMU) and grout. Where it is necessary to remove and replace grouted concrete masonry units, there might not be adequate access to recast grout in the cells. One approach would be to use wider masonry units, so the top of the unit extends a distance beyond the foundation sill plate that the foundation is supporting. Another approach is to split the CMU blocks in the longitudinal direction and use a half-block facing outward as a fascia to provide the appearance of masonry, while casting concrete against the back face.

4.4 Sill Plates and Anchorage

This section addresses wood (foundation) sill plates and the anchorage of a house to a concrete or masonry foundation. Section 4.4.1 through Section 4.4.5 provide repair recommendations for sill plates and anchorage. Section 4.4.6 addresses repair methods for sill plates and anchorage.

4.4.1 Wood Sill Plates

Sill plates sitting directly on a concrete or masonry foundation (i.e., foundation sill plates) are generally supporting wall studs or floor joists. Sill plates are typically of decay-resistant material (either preservative treated or of species of natural resistance to decay) and typically but not always anchored to the foundation with anchor bolts or other anchorage devices.

Repairs to wood sill plates are to be determined based on the earthquake damage patterns provided in Table 4-8. More detailed discussion of repair methods is provided in Section 4.4.6.

Table 4-8 Repair of Wood Sill Plates [1,2]

	Earthquake Damage Pattern	*Repair Method [3]*
4-8A	Sliding in any horizontal direction up to 1/4 inch with no other distress	No repair
4-8B	Local sill plate splitting at individual bolts, not more than one out of ten bolts	Add a replacement bolt or replacement retrofit plate in the vicinity of the fracture location for each affected bolt
4-8C	Sliding in any horizontal direction more than 1/4 inch	Move sill plate as close to its original location as possible. Install new anchorage to the foundation. Otherwise provide support and anchorage in the displaced location
4-8D	Sill plate splitting extending to more than one bolt in a row or at more than one out of ten bolts	Reinforce existing sill plate in place where possible, otherwise replace sill plate
4-8E	Local splitting not associated with anchor bolts	Where local splitting interferes with support of framing members above, remove and replace the section of sill. Where local splitting is away from members to be supported, place a 2x block over the split area and nail
4-8F	Splitting of blocking on top of foundation sill plate added as part of cripple wall retrofit	Remove sheathing, remove and replace blocks, replace sheathing
4-8G	Widespread splitting not associated with anchor bolts	Reinforce existing sill plate in place where possible, otherwise replace sill plate
4-8H	Crushing of the sill plate of not more than 1/8 inch of the sill plate under studs or posts	Shim at crushed location to regain bearing or provided sister stud alongside
4-8I	Crushing of the sill plate of more than 1/8 inch of the sill plate under studs or posts	Provide sister studs alongside existing to reestablish bearing

(1) Damage patterns with gray shading are also identified in the *General Guidelines*.

(2) Identify damage pattern and repair where damage has been identified during the damage investigation. Select damage pattern most representative of observed damage caused by or worsened by the earthquake. See Section 3.1.2 and Section 4.1.2 for discussion of worsened damage.

(3) See Section 4.4.6 for discussion of repair methods.

4.4.2 Steel Cast-In-Place Anchor Bolts

Repairs to steel cast-in-place anchors bolts are to be determined based on the earthquake damage patterns provided in Table 4-9. More detailed discussion of repair methods is provided in Section 4.4.6.

Table 4-9 Repair of Steel Cast-In-Place Anchor Bolts [1]

Earthquake Damage Pattern		Repair Method [2]
4-9A	Bending More than 15-degree angle or more than 1/4-inch gap between bolt and sill	Move sill plate back and add a replacement bolt or retrofit plate in the vicinity of the bent bolt location for each affected bolt
4-9B	Fracture Any observable indication of fracture	Add a replacement bolt or retrofit plate in the vicinity of the fracture location for each affected bolt
4-9C	Spalling More than 1-inch-deep spall immediately adjacent to anchor bolt	Add a replacement bolt or retrofit plate in the vicinity of the spall location for each affected bolt

[1] Identify damage pattern and repair where damage has been identified during the damage investigation. Select damage pattern most representative of observed damage caused by or worsened by the earthquake. See Section 3.1.2 and Section 4.1.2 for discussion of worsened damage.

[2] See Section 4.4.6 for discussion of repair methods.

4.4.3 Steel Post-Installed Anchor Bolts

Repairs to steel post-installed anchor bolts are to be determined based on the earthquake damage patterns provided in Table 4-10. More detailed discussion of repair methods is provided in Section 4.4.6.

Table 4-10 Repair of Steel Post-Installed Anchor Bolts [1]

Earthquake Damage Pattern		Repair Method [2]
4-10A	Bending More than 15-degree angle or more than 1/4-inch gap between bolt and sill	Move sill plate back and add a replacement bolt or retrofit plate in the vicinity of the bent bolt location for each affected bolt
4-10B	Fracture Any observable indication of fracture	Add a replacement bolt or retrofit plate in the vicinity of the fracture location for each affected bolt
4-10C	Spalling More than 1/2-inch-deep spall immediately adjacent to anchor bolt	Add a replacement bolt or retrofit plate in the vicinity of the spall location for each affected bolt
4-10D	Withdrawal or other damage to the wedging or undercutting mechanism	Add a replacement bolt or retrofit plate in the vicinity of the damaged bolt for each affected bolt

[1] Identify damage pattern and repair where damage has been identified during the damage investigation. Select damage pattern most representative of observed damage caused by or worsened by the earthquake. See Section 3.1.2 and Section 4.1.2 for discussion of worsened damage.

[2] See Section 4.4.6 for discussion of repair methods.

4.4.4 Steel Post-Installed Plate Connectors

Repairs to steel post-installed connector plates are to be determined based on the earthquake damage patterns provided in Table 4-11. More detailed discussion of repair methods is provided in Section 4.4.6.

Table 4-11 Repair of Steel Post-Installed Plate Connectors [1]

Earthquake Damage Pattern		*Repair Method* [2]
4-11A	Plate bending Such that plate might not function as intended	Install new retrofit plate in vicinity of damaged plate
4-11B	Plate fracture Any observable indication of fracture	Install new retrofit plate in vicinity of damaged plate
4-11C	Fastener to wood bending	Where possible remove and replace fasteners. Otherwise, install new retrofit plate in vicinity of damaged plate
4-11D	Fastener to wood fracture Any observable indication of fracture	Where possible remove and replace fasteners. Otherwise, install new retrofit plate in vicinity of damaged plate
4-11E	Fastener to concrete bending More than 15-degree angle or more than 1/4-inch gap between bolt and sill	Install new retrofit plate in vicinity of damaged plate
4-11F	Fastener to concrete fracture Any observable indication of fracture	Install new retrofit plate in vicinity of damaged plate
4-11G	Wood splitting To the extent that fastener capacity is reduced by more than 20 percent	Install new retrofit plate in vicinity of damaged plate
4-11H	Concrete spalling More than 1/2-inch deep spall immediately adjacent to anchor bolt	Install new retrofit plate in vicinity of damaged plate

(1) Identify damage pattern and repair where damage has been identified during the damage investigation. Select damage pattern most representative of observed damage caused by or worsened by the earthquake. See Section 3.1.2 and Section 4.1.2 for discussion of worsened damage.

(2) See Section 4.4.6 for discussion of repair methods.

4.4.5 Hillside Houses: Framing-to-Foundation Connections

Repairs for hillside house anchorage to the uphill foundation are to be determined based on the earthquake damage patterns provided in Table 4-12. More detailed discussion of repair methods is provided in Section 4.4.6.

Because detachment of the house from the uphill foundation can potentially result in collapse of the house, the intent of the repair is to return the anchorage of the house to uphill foundation to its pre-earthquake condition.

Table 4-12 Repair of Hillside House Anchorage to the Uphill Foundation (1)

	Earthquake Damage Pattern	*Repair Method* (2)
4-12A	Horizontal or vertical movement of house away from uphill foundation by not more than 1/2 inch	Support house in place and re-establish connection to the uphill foundation
4-12B	Horizontal or vertical movement of house away from the uphill foundation by more than 1/2 inch	Return house as close as practical to its original location and re-establish connection to the uphill foundation

(1) Identify damage pattern and repair where damage has been identified during the damage investigation. Select damage pattern most representative of observed damage caused by or worsened by the earthquake. See Section 3.1.2 and Section 4.1.2 for discussion of worsened damage.

(2) See Section 4.4.6 for discussion of repair methods.

4.4.6 Repair Methods for Wood Sill Plates and Anchorage

This section provides discussion of repair techniques applicable to wood sill plates and their anchorage, as discussed in Section 4.4.1 through Section 4.4.5.

Replacement anchor bolts. Where anchor bolts are to be replaced, one new bolt should be added for each bolt determined to not be effective. Replacement bolts can be epoxy anchored threaded rods, or where the concrete is strong enough to hold them, expansion anchors or concrete screws. Where the overhead height does not permit drilling for new anchors, retrofit anchors bolted to the face of the foundation can be used. Replacement anchor bolts are to be installed in accordance with the manufacturer's recommendations. Documents developed for voluntary seismic upgrade, as discussed in Appendix B, can provide useful guidance for installation of replacement bolts and retrofit plates.

When installing new anchor bolts, the typical bolt placement requirements of the applicable building or residential code should be followed to the extent possible. These requirements include minimum and maximum distances from the end of a foundation sill plate to the center of the anchor bolt, and minimum and maximum on-center spacing. Building and residential codes all require use of steel plate washers on anchor bolts for new construction. While not required for in-kind replacement, use of the plate washers is recommended.

Reinforcement of existing sill plates. Where existing sill plates have local splits, it may be practical to reinforce the sill plate in place; this can be a practical alternative to the onerous work of replacing the sill plate. For cracks in sill plates 1/32 inch or less in width, screws installed to stitch across the cracks should resist further crack spreading and maintain the capacity of adjacent bolts. For cracks in sill plates of greater than 1/32 inch, installation of a retrofit plate for each bolt can both provide screws to stitch across the crack and provide anchorage capacity to supplement the anchor bolts. One retrofit anchor should be added for each anchor bolt occurring at a split.

Replace sill plate. Removal and replacement of sill plates should only be specified where other less invasive repair methods are not adequate. Where sills are to be removed and replaced, the length to be removed and replaced should be long enough that at least two anchor bolts can be installed in the replacement piece. Removal and replacement of sills will generally require shoring of the wall or floor framing above. When sills are replaced, studs will need to be refastened to the new sill plate, using toe-nails or framing clips.

Move hillside house back to its original location. Repair intends that an attempt be made to move the house back to its original location. This may or may not require removal of cripple walls and similar elements, and replacement after the house has been moved.

Re-establish hillside house connection to uphill foundation. When hillside houses move away from the uphill foundation, splitting of the uphill foundation sill plate or ledger is likely to occur. Repair of this and other associated damage is to be provided in accordance with applicable sections of this document.

4.5 Wood-Frame Walls

Wood-frame walls include common site-built wall elements, their sheathing and finish materials, framing, tie-downs and load-path elements, and other vertical elements of the gravity-load-carrying system. Other seismic-force-resisting elements (prescriptive pre-engineered, engineered, and proprietary) are discussed in Section 4.6. When repairing wood-frame walls, all applicable elements and damage patterns should be considered. For example, when damage to diagonal lumber sheathing has associated damage to the supporting framing, both Section 4.5.8 (diagonal lumber sheathing) and Section 4.5.10 (framing) should be consulted.

This section discusses sheathing and finish materials that are most commonly used in houses and that contribute meaningful strength and stiffness. Other exterior finish materials are used in houses, including wood shingles; hardboard, aluminum, vinyl, and lap siding; exterior insulation and finish systems (EIFS); and anchored and adhered veneer. These are generally thought to contribute little to the strength and stiffness of houses and are beyond the intended scope of the *Engineering Guidelines*.

The intent of these repairs is to substantially restore strength and stiffness. For some materials, the repairs specifically draw from tests conducted to verify that stiffness and strength are substantially restored. For other materials, common repair practice, believed to achieve this goal, is provided. As a general philosophy, damage to wood wall framing and sheathing materials should, whenever possible, be addressed by leaving existing elements in place and supplementing with new as required. Where both removal and replacement, and supplementing existing construction are viable alternatives, supplementing should be favored. This can help to avoid both disruption to surrounding elements and the need for shoring during removal.

Repairs are applicable to both the walls in the occupied stories of a house and the cripple walls surrounding unoccupied crawlspaces. In concept there is nothing different regarding repair of a cripple wall versus an occupied story wall, except that access from the crawlspace is different, and higher levels of drift and damage can be concentrated in cripple walls. Resources developed for seismic retrofit of cripple walls, as discussed in Appendix B, might be of use in identifying repairs to damaged cripple walls.

4.5.1 Horizontal Wood Siding

Horizontal wood shiplap siding includes siding in which the siding sits flush against and is nailed to the supporting wall framing. Lap siding and similar siding products that are installed at an angle to the wall framing serve only as an exterior finish and not as bracing material and are not addressed in the *Engineering Guidelines*. See the Glossary for types of horizontal wood siding.

Repairs to horizontal wood siding are to be determined based on the earthquake damage patterns provided in Table 4-13, and the more detailed descriptions that follow.

Table 4-13 Repair of Horizontal Wood Shiplap Siding

Earthquake Damage Pattern		*Repair Method*
4-13A	Nail head withdrawal	Reset nails, patch and refinish
4-13B	Local splitting of siding at nails (particularly at end of siding board)	Renail siding at splits, patch, and refinish
4-13C	Corner trim loosened or fractured	Check for and correct any residual drift, replace or refasten corner trim, patch and paint
4-13D	Gap between siding and supporting framing of up to 1/4 inch	Check for and correct any residual drift, reset existing nails, add new nails as required, patch and paint
4-13E	Studs are observed to be locally split at many siding nails, back of siding is deformed around nail holes, siding nails are easily withdrawn	Renail siding
4-13F	Building paper or other weather resistive barrier (WRB) has more than limited tearing	Remove and replace siding and building paper

Notes: (a) Damage patterns with gray shading are also identified in the *General Guidelines*.
(b) Identify damage pattern and repair where damage has been identified during the damage investigation. Select damage pattern most representative of observed damage caused by or worsened by the earthquake. See Section 3.1.2 and Section 4.1.2 for discussion of worsened damage.

In houses with horizontal wood siding and cripple walls, significant racking can occur in the cripple walls while damage to the wood siding remains minimal. See Section 4.5.10 for discussion of wall racking damage and repair.

Reset nails. Where heads of nails protrude, reset as required to patch and paint. Where resetting of a nail is not possible, it should be replaced.

Renail siding. Renail siding, patch, and paint. This will reattach the wood siding as required for an exterior finish.

Remove and replace siding. Siding is to be removed, drift addressed if required, building paper removed and replaced, and siding replaced. Often siding will get damaged during removal, and new siding should be installed. Removal and reinstallation is acceptable where possible. Depending on the extent of stud splitting at nail holes, installation of new sister studs might be required for nailing of siding. Existing studs should be left in place and new studs provided alongside and inter-nailed.

4.5.2 Plywood Panel Siding

Plywood panel siding repairs are to be determined based on the earthquake damage patterns provided in Table 4-14. The damage patterns and repairs are most similar to those for wood structural panel sheathing but also include considerations of the building envelope and weather resistance.

Plywood panel siding is a subset of plywood that is specifically manufactured for use as an exterior finish material. Plywood panel siding has been extensively used in house construction since the 1960s. When used in conventional construction of houses, the siding serves as both siding and seismic bracing. When used in engineered houses, it is possible but not common that there be wood structural panel sheathing installed underneath the panel siding. Plywood panel siding is generally installed in 4-foot-wide sheets with heights between 8 feet and 10 feet. A repeated mis-installation is found in western states where this product is prevalent. The mis-installation occurs at abutting panel edges, where only one of the two abutting panel edges is appropriately edge nailed (Figure 4-2). Where this occurs, the panels are particularly vulnerable to damage.

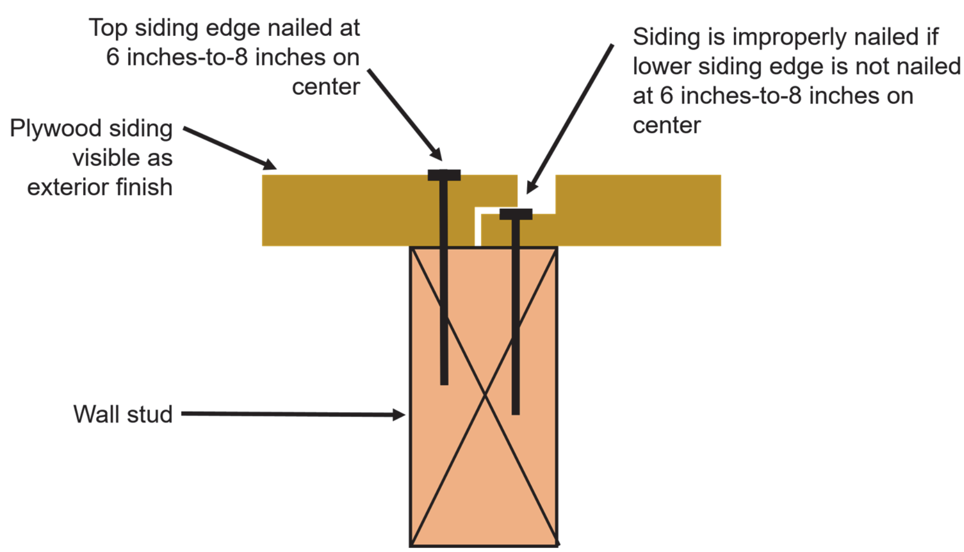


Figure 4-2 Common mis-installation of wood structural panel sheathing at abutting panel edges.

Table 4-14 Repair of Plywood Panel Siding

	Earthquake Damage Pattern	*Repair Method*
4-14A	Nail head withdrawal without significant enlargement or tearing of the panel around the nail	Reset nails. If nail does not properly reset, add one new nail for each withdrawn nail. If originally painted, patch and refinish
4-14B	Nail head withdrawal with significant enlargement or tearing of hole around the nail	Renail siding, relocating nails to undamaged area, patch, and refinish
4-14C	Disruption of flashing at horizontal joints	Refasten or replace flashing
4-14D	Tearing of siding sheets or tearing of nails through edge of panel. Gap between siding and supporting framing. Rotation of plywood panel to more than 1/4 inch out of vertical on any side	Remove and replace siding and building paper
4-14E	Where siding is removed, stud is observed to be locally split at most siding nails, or building paper is torn	Siding to be removed, building paper removed and replaced, and siding replaced. Often siding will get damaged during removal, and new siding should be installed. Removal and reinstallation is acceptable where possible. Depending on the extent of stud splitting at nail holes, installation of new sister studs might be required for nailing of siding. Existing studs should be left in place and new studs provided alongside and inter-nailed
4-14F	Building paper or other WBR has more-than-limited tearing	Remove and replace building paper and siding.

Notes: [a] Damage patterns with gray shading are also identified in the *General Guidelines*.

[b] Identify damage pattern and repair where damage has been identified during the damage investigation. Select damage pattern most representative of observed damage caused by or worsened by the earthquake. See Section 3.1.2 and Section 4.1.2 for discussion of worsened damage.

Reset nails. Where heads of nails protrude, reset as required to patch and paint. Where resetting of a nail is not possible, it should be replaced.

Renail siding. Renail siding, patch, and paint. This will reattach the wood siding as required for an exterior finish.

Remove and replace siding. Siding is to be removed, drift addressed if required, building paper removed and replaced, and siding replaced. Often siding will get damaged during removal, and new siding should be installed. Removal and reinstallation is acceptable where possible. Depending on the extent of stud splitting at nail holes, installation of new sister studs might be required for nailing of siding. Existing studs should be left in place and new studs provided alongside and inter-nailed.

4.5.3 Stucco

This section addresses repair to exterior stucco. Use of exterior insulation and finishing systems (EIFS) is seen occasionally, particularly in recently constructed housing developments. It is important that this be differentiated from stucco. EIFS serves only as an exterior finish and not a bracing material, and it is not addressed by the *Engineering Guidelines*.

Stucco repairs are to be determined based on the earthquake damage patterns provided in Table 4-15 and the more detailed descriptions that follow.

Table 4-15 Repair of Stucco

Earthquake Damage Pattern		Repair Method
4-15A	Crack in stucco Width: up to 1/64 inch	No repair
4-15B	Crack in stucco Width: up to 1/8 inch Detachment from framing or spalling: slight to none	Rout, patch, refinish
4-15C	Crack in stucco Width: up to 1/8 inch Pattern: extensive, see Figure 4-3 Spalling or offsets: slight to none	Remove finish coat, rout, patch and refinish
4-15D	Cracking in stucco Width: more than 1/8 inch limited to wall corners	Remove stucco, wire mesh and building paper at wall corners plus a distance far enough to lap building paper and wire, and reinstall
4-15E	Stucco spall: locally at cracks Stucco wire: fractured locally	Remove stucco at affected crack areas plus a distance far enough to lap building paper and wire. Replace building paper or other WRB and reinstall stucco
4-15F	Stucco has detached from underlying sheathing or framing, separating by more than 1/8 inch, or stucco has out-of-plane offsets at cracks or joints of more than 1/16 inch	Remove stucco at affected story level following wall plane until all loosened stucco has been removed. Replace building paper or other WRB and reinstall stucco
4-15G	Building paper or other WRB has more than limited tearing	Remove and replace building paper and stucco

Notes: [a] Damage patterns with gray shading are also identified in the *General Guidelines*.

[b] Identify damage pattern and repair where damage has been identified during the damage investigation. Select damage pattern most representative of observed damage caused by or worsened by the earthquake. See Section 3.1.2 and Section 4.1.2 for discussion of worsened damage.

Stucco can withstand fairly widespread cracking without notable loss of strength or stiffness. Where notable loss of strength and stiffness occurs, it is primarily due to detachment of stucco fasteners from supporting framing (e.g., nail head pull out from stucco, staple withdrawal from framing). At larger drift levels, spalling of stucco and breaking of lath wires occurs. Stucco on cripple walls has behavior similar to walls of occupied stories but can often tolerate higher levels of drift. In cripple wall houses, the level of damage and needed level of repair required for the cripple wall might be different than for the walls of the occupied stories above. Where stucco on the cripple wall is damaged, it is often possible to repair or replace the cripple wall stucco while leaving the stucco above in place. This extent of repair is appropriate for the commonly occurring condition where the stucco becomes detached from the cripple wall foundation sill plate.

The repairs that follow were in large part derived from testing by Arnold et al. (2003), discussed in Appendix D, Section D.2. This testing included demonstration that the described repairs could substantially return strength and stiffness to damaged stucco.

Fine cracks (i.e., 1/64 inch or narrower) should not be patched, especially if the stucco is not painted (i.e., integral color stucco). On painted stucco, cracks this fine will be sealed by a fresh coat of paint. When determining the area to be painted, consideration should be given to obtaining a reasonably uniform appearance.

For cracks up to 1/8-inch wide, the crack should be opened to the brown coat by beveling the crack edges to accept patching material. Patch with flexible vinyl base patching compound. Stucco should be applied to match the existing surface texture as necessary. The stucco should be refinished as necessary to match adjacent areas. When determining the area to be refinished, consideration should be given to obtaining reasonably uniform appearance.

When the number of cracks to be patched becomes extensive, it may be more economical to apply a new finish coat. The process of sandblasting will accentuate cracks visible in the finish coat and expose cracks in the brown coat. When determining whether observed cracks are earthquake related, it is important to note that cracks exposed by the sandblasting are shrinkage cracks that date from original application of the stucco and not earthquake cracking; a stucco finish coat, even if painted with latex paint, has no ability to conceal earthquake cracking of stucco. When determining the area to be refinished, consideration should be given to obtaining a reasonably uniform appearance. Figure 4-3 provides an example of cracking patterns that might be considered "extensive." This is to provide a general indication; actual extent and pattern of cracking can vary.

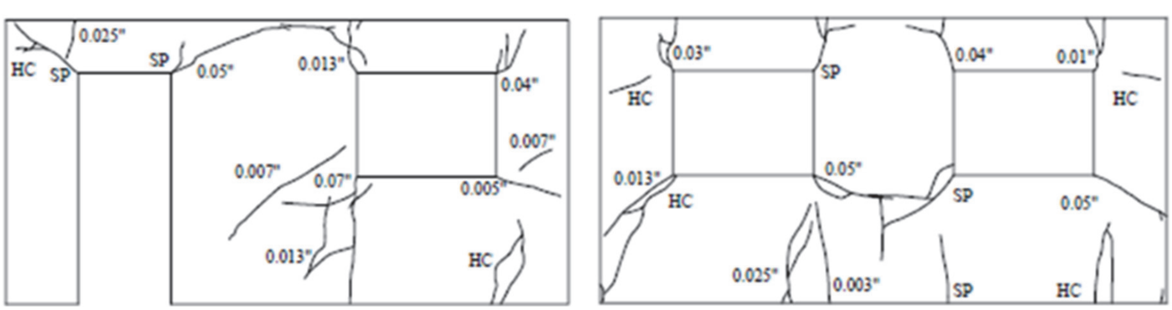


Figure 4-3 Example of extensive cracking.

Where the stucco has buckled or detached from the framing, or is severely cracked, the existing stucco should be removed back to intact and securely attached stucco. Where detachment has occurred, it will often extend the full length of the wall line for the affected story level. The underlying building paper should be repaired or replaced as necessary, and any new paper should be properly lapped with the existing building paper. New wire mesh should be installed and nailed to the framing, and it should overlap existing mesh by at least 6 inches. Stucco should be applied in three coats to match the existing thickness and surface finish. The stucco should be refinished as necessary to match adjacent areas. When determining the area to be refinished, consideration should be given to obtaining a reasonably uniform appearance.

Typical stucco is applied in three coats, with time permitted for drying between each of these coats. Although it is possible for stucco to delaminate (i.e., to have separations between coats), this is not typically seen and not likely related to earthquake loading.

4.5.4 Plaster on Wood Lath

This section addresses repair of plaster on wood lath installed as a wall interior finish. Plaster on wood lath repairs are to be determined based on the earthquake damage patterns provided in Table 4-16, and the detailed discussions that follow.

Table 4-16 Repair of Plaster on Wood Lath

Earthquake Damage Pattern		*Repair Method*
4-16A	Limited cracks up to 1/64-inch wide	Patch and refinish
4-16B	Limited cracks up to 1/8-inch wide	Rout, patch, and refinish
4-16C	Local compression buckling of finish coat	Remove plaster locally at spall, route, patch and paint
4-16D	Extensive cracking or spalling, or cracks more than 1/8-inch wide	Remove plaster and wood lath and replace with gypsum wallboard over wood structural panel sheathing

Notes: [a] Damage patterns with gray shading are also identified in the *General Guidelines*.
[b] Identify damage pattern and repair where damage has been identified during the damage investigation. Select damage pattern most representative of observed damage caused by or worsened by the earthquake. See Section 3.1.2 and Section 4.1.2 for discussion of worsened damage.

Replace with gypboard over wood structural panel sheathing. This repair is intended to restore strength and at the same time provide the same total thickness as the plaster and wood lath that was removed. This allows existing trim around windows and doors to be reused without having to make potentially costly adjustments. The total thickness of plaster and wood lath can vary but is commonly about 7/8 inch. Restoring this thickness will commonly require use of 3/8-inch plywood and 1/2-inch gypboard panels. The actual existing material thickness will need to be determined when repairs are implemented, and the thickness of the plywood sheathing adjusted accordingly. Plywood sheathing should have all edges blocked and be nailed with not less than 8-penny common nails spaced not more than 4-inches on center at panel edges and 12-inches on center at other supports.

4.5.5 Plaster on Gypsum Lath

Plaster on gypsum lath installed as a wall interior finish uses a finish coat of plaster on a gypsum board base. The gypsum board base generally has regularly spaced holes that allow the plaster to key onto the board. It is sometimes referred to as "button board" because of the plaster buttons that key into the holes. This interior finish material combines behaviors of plaster and of gypsum wallboard.

Plaster on gypsum lath repairs are to be determined based on the earthquake damage patterns provided in Table 4-17. These descriptions are integrated with the *General Guidelines*.

Table 4-17 Repair of Plaster on Gypsum Lath

Earthquake Damage Pattern		*Repair Method*
4-17A	Short cracks up to 1/64-inch wide	Patch and refinish
4-17B	Cracks up to 1/8-inch wide, no detachment of gypsum lath, no spalling	Rout, patch, and refinish
4-17C	Shifting and detachment of gypsum lath or cracks more than 1/8-inch wide	Remove plaster and gypsum lath and replace with gypsum wallboard over a gypsum wallboard or wood structural panel sheathing base as required to match existing thickness

Notes: [a] Damage patterns with gray shading are also identified in the *General Guidelines*.
[b] Identify damage pattern and repair where damage has been identified during the damage investigation. Select damage pattern most representative of observed damage caused by or worsened by the earthquake. See Section 3.1.2 and Section 4.1.2 for discussion of worsened damage.

Where the plaster is cracked but remains firmly attached to the gypsum lath, repairs can be accomplished by cleaning the crack, and patching, texturing, and painting to match the existing surface texture and finish. When determining the area to be retextured and repainted, consideration should be given to obtaining a reasonably uniform appearance.

Where the lath has fractures (or where plaster damage suggests fracture of the gypsum lath), the damaged pieces should be removed to soundly attached plaster or lath or both, new lath installed, and the area replastered. Where large areas of repair are involved and it is more economical to do so, plaster and lath should be removed to the limits of the wall panel and replaced with gypsum wallboard. When determining the area to be refinished, consideration should be given to obtaining a reasonably uniform appearance.

4.5.6 Gypsum Wallboard

Repairs to gypsum wallboard (also known as gypboard or drywall) installed as a wall interior finish are to be determined based on the earthquake damage patterns provided in Table 4-18 and the detailed repair descriptions that follow. These descriptions are integrated with the *General Guidelines*.

Table 4-18 Repair of Gypsum Wallboard

Earthquake Damage Pattern		*Repair Method*
4-18A	Short cracks up to 1/64-inch wide	Path and refinish
4-18B	Cracks following taped joints or corner bead, puckering or buckling of tape at joint	Re-tape joints as required, rout, patch, and refinish
4-18C	Nail pops	Add drywall screw 1 inch from original fastener, set or remove original fastener, patch, and refinish
4-18D	Cracks up to 1/8-inch wide through the thickness of the board	Remove and replace gypboard to nearest studs beyond crack (minimum 32 inches by 48 inches), refinish
4-18E	Crushing and puckering of gypboard at panel edges	Remove and replace gypboard to nearest studs beyond damage (minimum 32 inches by 48 inches), refinish
4-18F	Tear-out of fasteners at panel edges	Remove and replace gypboard to nearest studs beyond damage (minimum 32 inches by 48 inches), refinish
4-18G	Tape at panel joints is torn or separated Back face of board is slotted by fastener to maximum length of 1/2 inch	Add drywall screw 2 inches from original fastener, set or remove original fastener, patch and refinish
4-18H	Back face of board is slotted by fastener to length greater than 1/2 inch, or board is visibly rotated	Remove and replace gypboard to nearest studs beyond damage (minimum 32 by 48 inches), refinish
4-18I	Gypboard is detached from wall framing.	Remove and replace entire board, tape and finish.

Notes: [a] Damage patterns with gray shading are also identified in the *General Guidelines*.
[b] Identify damage pattern and repair where damage has been identified during the damage investigation. Select damage pattern most representative of observed damage caused by or worsened by the earthquake. See Section 3.1.2 and Section 4.1.2 for discussion of worsened damage.

Where cracking follows panel joints or corner beads, existing tape and compound should be removed. The joint should then be retaped, retextured, and repainted. When determining the area to be retextured and repainted, consideration should be given to obtaining a reasonably uniform appearance.

Short (less than about 6-inches long) cracks less than about 1/64-inch wide and extending from the corners of openings may be patched using drywall tape and joint compound, and then retextured and repainted. When determining the area to be retextured and repainted, consideration should be given to obtaining a reasonably uniform appearance

Where cracks are up to about 1/8-inch wide and extend through the thickness of the drywall, the cracked piece should be removed to the nearest stud on either side of the crack (32-inch minimum width of removed gypboard) and replaced, retextured, and repainted.

Nail pops may be repaired by adding a drywall screw adjacent to the nail pop, resetting or removing the popped fastener, patching, retexturing and repainting. When determining the area to be retextured and repainted, consideration should be given to obtaining a reasonably uniform appearance.

Fractured gypsum wallboard panels should be replaced in kind. Where the attachment of the drywall to the framing has loosened significantly, new fasteners should be installed around the wall perimeter and along the panel joints showing signs of relative movement. The repair areas should be refinished as necessary to match adjacent areas. When determining the area to be retextured and repainted, consideration should be given to obtaining a reasonably uniform appearance. Where gypsum wallboard is replaced, it is important that the panel thickness and type (e.g., Type X) of the replacement panel match the original construction.

4.5.7 Wood Structural Panel, Particleboard, and Fiberboard Sheathing

Repairs to wood structural panel wall sheathing (plywood and oriented strand board, or OSB) and particleboard and fiberboard sheathing are to be determined based on the earthquake damage patterns provided in Table 4-19 and the more detailed descriptions that follow.

Table 4-19 Repair of Wood Structural Panel, Particleboard, and Fiberboard Sheathing

Earthquake Damage Pattern		*Repair Method*
4-19A	Nail-head withdrawal without significant enlargement or tearing of the panel around the nail	Reset nails, add nails where cannot be reset
4-19B	Nail-head withdrawal with significant enlargement or tearing of hole around the nail or fracture of the panel face due to the nail digging in (like overdriving of nails)	Renail sheathing
4-19C	Tearing of panels or tearing of nails through edge of panel Sheathing panel rotation Gap between panels and supporting framing Panels detached from supporting framing	Remove and replace sheathing
4-19D	At investigation opening where sheathing is removed, stud or other framing to which the sheathing is fastened is observed to be locally split	Remove and replace sheathing

Note: Identify damage pattern and repair where damage has been identified during the damage investigation. Select damage pattern most representative of observed damage caused by or worsened by the earthquake. See Section 3.1.2 and Section 4.1.2 for discussion of worsened damage.

Wood structural panel sheathing in cripple walls has been observed in testing to be able to withstand higher drift ratios (drift divided by wall height) than full-story height walls. For this reason, cripple walls that have experienced significant drift ratios might have less damage than expected.

Reset nails. If nail does not properly reset, add one new nail for each withdrawn nail.

Renail sheathing. Where only a limited number of nails is involved or the existing edge-nail spacing is at 6-or-more inches on center, it is possible to simply add additional nails to the existing sheathing. The added nails would typically be centered between existing nails, with staggering of nail lines provided where possible. Where existing nailing is closer than 4-inches on center, predrill holes for renailing. Where existing nailing is 2-inches on center or closer, remove panels, sister existing studs with new, and reinstall panels with staggered nailing.

Remove and replace sheathing. Once tearing of the sheathing occurs, or the panel is detached from and gaps from the framing, it is appropriate to remove and replace the sheathing. Depending on the extent of stud splitting at nail holes, installation of new sister studs might be required for nailing of new sheathing. Existing studs should be left in place, and new studs provided alongside and inter-nailed. Repair to studs is addressed in Section 4.5.10.

4.5.8 Diagonal Lumber Sheathing

Repairs to diagonal lumber sheathing are to be determined based on the earthquake damage patterns provided in Table 4-20, and the more detailed descriptions that follow.

Table 4-20 Repair of Diagonal Lumber Sheathing

Earthquake Damage Pattern		*Repair Method*
4-20A	Hairline splitting of boards at nail locations	No repair
4-20B	More than hairline splitting of boards at limited nail locations	Renail existing sheathing relocating nails where possible, where not possible replace sheathing board and renail
4-20C	More than hairline splitting of studs at numerous nail locations	Remove diagonal sheathing boards and replace with wood structural panel sheathing of equivalent thickness
4-20D	Limited splitting of studs, top plates, or bottom plates at sheathing nail locations	Add new nails between sheathing and framing away from splits
4-20E	Extensive splitting or fracture of studs, top plates, or bottom plates at sheathing nail locations	Where practical, provide sistered studs and other framing to allow renailing of existing sheathing. Where not practical, replace studs, top plates or bottom plates as required.

Note: Identify damage pattern and repair where damage has been identified during the damage investigation. Select damage pattern most representative of observed damage caused by or worsened by the earthquake. See Section 3.1.2 and Section 4.1.2 for discussion of worsened damage.

Remove and replace split sheathing boards. Where the damage only affects a few boards, it will be most expeditious to remove and replace those sheathing boards. Where large areas are being replaced, wood structural panel of equivalent thickness should be substituted. Wood structural panel nailing should be specified to obtain matching structural capacity. In instances where an entire wall surface is being repaired, it may be possible to leave existing diagonal sheathing in place and apply wood structural panel sheathing over; the effect of the additional thickness on door and window trim should be checked before proceeding with this repair.

Add new nails away from splits. Diagonal lumber sheathing is most often nailed with two nails at each location where the lumber sheathing crosses a framing member. Because the diagonal lumber sheathing is primarily working in tension and compression, the location of the nails in the area of crossing is not extremely sensitive. The nails should be located to best avoid splitting and be predrilled if necessary.

Replace framing. Where splitting of framing has occurred to the extent that renailing is not possible, replacement of framing will be required. Shoring may be required for framing replacement. See additional notes in Section 4.4 regarding replacement of foundation sill plates.

4.5.9 Horizontal Lumber Sheathing

Repairs to horizontal lumber sheathing are to be determined based on the earthquake damage patterns provided in Table 4-21, and the more detailed descriptions that follow.

Table 4-21 Repair of Horizontal Lumber Sheathing

Earthquake Damage Pattern		*Repair Method*
4-21A	Nail head withdrawal	Reset nails or renail boards
4-21B	Local splitting of boards at nails	Renail boards, relocating nails as required
4-21C	Extensive splitting or fracture of sheathing boards	Remove sheathing boards and replace with wood structural panel sheathing of equivalent thickness
4-21D	Gap between board and supporting framing of up to 1/4 inch	Renail boards
4-21E	Where board is removed, stud is observed to be locally split at a limited number of sheathing nails	Renail boards
4-21F	Where board is removed, stud is observed to be locally split at most sheathing nails, back of sheathing is deformed around nail holes, siding nails are easily withdrawn	Remove and replace sheathing with wood structural panel sheathing of equivalent thickness. Sister or replace split framing

Note: Identify damage pattern and repair where damage has been identified during the damage investigation. Select damage pattern most representative of observed damage caused by or worsened by the earthquake. See Section 3.1.2 and Section 4.1.2 for discussion of worsened damage.

Horizontal lumber sheathing is most commonly found on houses constructed before 1950. 1× sheathing of 6-inches nominal or wider is nailed directly to wall studs and is often overlain by building paper that provides a moisture barrier. Horizontal lumber sheathing is commonly found underneath stucco and shingle exterior finishes.

Remove and replace sheathing boards. Where the damage only affects a few boards, it will be most expeditious to remove and replace those sheathing boards. Where large areas are being replaced, wood structural panel of equivalent thickness should be substituted. Wood structural panel nailing should be specified to obtain matching structural capacity. In instances where an entire wall surface is being repaired, it may be possible to leave existing diagonal sheathing in place and apply wood structural panel sheathing over; the effect of the additional thickness on door and window trim should be checked before proceeding with this repair.

Add new nails away from splits. Horizontal lumber sheathing is most often nailed with two nails at each location where the lumber sheathing crosses a framing member. Because the lumber sheathing nailing is forming a couple that resists racking loads, the sheathing capacity in sensitive to the location of the nails in the area of crossing. Where is not possible to add new nails with a center-to-center spacing similar to the existing nails, it may be necessary to remove the sheathing and add new framing below to receive the nails. The nails should be predrilled as necessary.

Sister or replace framing. Where splitting of framing has occurred to the extent that renailing is not possible (i.e., nails cannot be located in the general vicinity of their intended locations), either sistering on new framing or replacement of framing will be required. Shoring may be required for framing replacement. See additional notes in Section 4.4 regarding replacement of foundation sill plates.

4.5.10 Framing

This section addresses damage to wall framing—including racking, leaning or other distortion; separation or sliding; and other damage (e.g., splitting, crushing). While this is primarily applicable to wall studs, also included are headers, top and bottom plates, and other miscellaneous framing that make up the wall framing system. See Section 4.5.12 for other vertical gravity-load-carrying elements that are not part of the wall system.

Wall Racking, Leaning, or Other Distortion

This section discusses racking, leaning, and other distortion of wall framing; distortion of wall finish and sheathing materials are addressed in other portions of Section 4.5. Repairs to wall racking, leaning, and other distortion are to be determined based on the earthquake damage patterns provided in Table 4-22, and the more detailed descriptions that follow.

Table 4-22 Repair of Racked or Leaning Framing

Earthquake Damage Pattern		*Repair Method*
4-22A	Racking or leaning of less than 1/2 inch over the height of a story or the height of a cripple wall	No repair
4-22B	Racking or leaning of more than 1/2 inch over the height of a story or cripple wall	Move the house as close to the original plumb position as practical. Repair sheathing and finish materials as required by other portions of Section 4.5
4-22C	Racking or leaning of more than 2 inches over the height of a story or cripple wall	Remove sheathing and finish materials, square up walls, refasten wall framing connections, and reinstall sheathing and finish materials

Note: Identify damage pattern and repair where damage has been identified during the damage investigation. Select damage pattern most representative of observed damage caused by or worsened by the earthquake. See Section 3.1.2 and Section 4.1.2 for discussion of worsened damage.

Where wall framing is leaning, racking, or otherwise distorted, this can lead to the need to remove all finish and sheathing materials and either move the wall framing back into plumb or reframe the wall. Because this could include removal of all finish and sheathing materials, evaluation of this behavior should be considered prior to detailed consideration of repairs to other wall elements. Observed wall distortion resulting from an earthquake can be due to earthquake loading or to movement of the foundation and soils that support the house. Where distortion is due to the movement of soils or foundations, the evaluation and repair should consider the best approach to addressing the combined damage.

Racking or leaning less than 1/2 inch per story. Tolerances are inherent in any construction type and very small racking, leaning, and other distortion can be an indication of original construction tolerances or other causes, such as seasonal soil swelling and drying, soil densification, or soil creep. This level of racking and leaning is anticipated to be widely occurring in the existing building stock prior to an earthquake.

Racking or leaning increased by more than 1/2 inch per story. Wall racking or leaning of 1/2 inch over a story height of 8 feet to 10 feet is thought to be a modest level, at which the leaning or racking will just start to be noticeable, and damage in the sheathing and wall finish materials should be limited. The same 1/2 inch would be considered similarly if occurring over the height of a cripple wall. For this level of racking or leaning, practical measures should be taken to reduce the level of rack or lean and secure existing sheathing and wall finish materials. Removal and replacement of finishes is not required unless triggered by other provisions of this guideline.

Racking or leaning of more than 2 inches per story. Wall racking or leaning of 2 inches or more over a story height of 8 feet to 10 feet is thought to be a significant level at which the leaning or racking will be quite noticeable, may affect the function of doors and windows, and notable damage would be anticipated to occur in the sheathing and wall finish materials, and possibly in the framing. The same 2 inches would be considered similarly if occurring over the height of a cripple wall. For this level of racking or leaning, repair will generally involve removal of all finish and sheathing materials and either moving ("squaring") the wall framing back into plumb or reframing the wall. Two exceptions to this are horizontal lumber siding and horizontal lumber sheathing; for these materials it may sometimes be possible to re-square the framing and refasten the boards without removing and replacing them. The choice to plumb the wall or reframe the wall should be made based on presence of other framing damage that might dictate reframing, and relative cost and practicality of the two options. Occasionally local crushing of wall top or bottom plates can occur due to the racking or leaning of the studs. Where this occurs, it can be addressed by adding new sister studs alongside existing, with local filling or shimming of the depressions, or by local replacement of the foundation sill plate.

A house falling completely off of the cripple walls is an extreme example of racking and leaning. In cases where the house has sustained limited damage, it is feasible to lift the house back to the intended location, shore it in place, and reconstruct cripple walls below; this occurs most often when the house falls as a unit onto level ground. In other cases, the damage to the house can be more extensive, possibly requiring complete replacement of the house; this occurs most often when the house falls unevenly or falls to sloped ground. In these cases, severe racking of the house superstructure can occur. The extent of damage will need to be assessed to determine the best approach for repair.

Framing Separation, Sliding, Unseating, Rocking, Gapping

Repairs to framing members that have become displaced due to earthquake loading, including framing separation, sliding, unseating, rocking, or otherwise gapping are to be determined based on the earthquake damage patterns provided in Table 4-23, and the more detailed descriptions that follow.

Table 4-23 Repair of Framing Displacement

Earthquake Damage Pattern		*Repair Method*
4-23A	Slip of floor framing relative to foundation sill plate of more than 1/4 inch in any direction	Return framing to the intended location to the extent possible and re-establish fastening
4-23B	Slip of studs relative to foundation sill plate or bottom plate of more than 1/4 inch in any direction	Return framing to the intended location to the extent possible and re-establish fastening
4-23C	Slip of bottom plate relative to floor sheathing of more than 1/4 inch in any direction	Return framing to the intended location to the extent possible and re-establish fastening
4-23D	Slip of studs relative to top plate of more than 1/4 inch in any direction	Return framing to the intended location to the extent possible and re-establish fastening
4-23E	Slip between upper and lower top plate of more than 1/4 inch in any direction	Return framing to the intended location to the extent possible and re-establish fastening
4-23F	Slip between top plate and rim joist or blocking of more than 1/4 inch in any direction	Return framing to the intended location to the extent possible and re-establish fastening
4-23G	Rocking of studs such that they are bearing on a corner rather than the full stud area	Return framing to the intended location to the extent possible and re-establish fastening

Table 4-23 Repair of Framing Displacement (continued)

Earthquake Damage Pattern		*Repair Method*
4-23H	Gapping of framing more than 1/4 inch that is likely to close under future gravity of lateral loading	Return framing to the intended location to the extent possible and re-establish fastening
4-23I	Unseating of beams on posts	Return framing to the intended location to the extent possible and re-establish fastening

Note: Identify damage pattern and repair where damage has been identified during the damage investigation. Select damage pattern most representative of observed damage caused by or worsened by the earthquake. See Section 3.1.2 and Section 4.1.2 for discussion of worsened damage.

Return framing to the intended location. Where slip of more than 1/4 inch has occurred, practical measures should be taken to return the framing as close as possible to its intended location.

Re-establish fastening. Most framing is fastened with full-length nails and can withstand some level of bending and withdrawal with moderate reduction in connection capacity. Once a slip of 1/4 inch has occurred, the connection strength will be reduced such that refastening should occur. For houses that do not have engineered designs, the refastening should be in accordance with the minimum fastening tables for the applicable building or residential codes, or framing clips that provide similar levels of connection. For engineered houses, the fastening should be per the original construction plans or be engineered.

Framing Damage

This section addresses repair of wood studs and related wall framing, including headers, top and bottom plates, and other miscellaneous wall framing. Wood stud and framing repairs are to be determined based on the earthquake damage patterns provided in Table 4-24 and the more detailed descriptions that follow.

Table 4-24 Repair of Other Framing Damage

Earthquake Damage Pattern		*Repair Method*
4-24A	Localized splitting of framing such that load carry capacity is not significantly impacted	Re-establish fastening, if required
4-24B	Extensive splitting such that load carrying capacity is affected or connections are disrupted	Sister framing where possible, replace framing otherwise
4-24C	Localized crushing not more than 1/8 inch	No repair
4-24D	Crushing more than 1/8 inch	Place sister studs on one or both sides, shim or fill crushed area, or replace framing

Note: Identify damage pattern and repair where damage has been identified during the damage investigation. Select damage pattern most representative of observed damage caused by or worsened by the earthquake. See Section 3.1.2 and Section 4.1.2 for discussion of worsened damage.

House framing can sustain a variety of damage types when subjected to earthquake loading, including splitting and crushing. While this is most likely to occur in elements of the seismic-force-resisting load path, damage can also occur in other members due to movement of the house. This can result in a shifting of gravity loads from their typical load path.

Re-establish fastening if required. Most framing is fastened with full-length nails and can withstand some level of bending and withdrawal with moderate reduction in connection capacity. Once a slip of 1/4 inch has occurred, the connection strength will be reduced such that refastening should occur. For houses that do not have engineered designs, the refastening should be in accordance with the minimum fastening tables from the applicable building or residential codes. For engineered houses, the fastening should be per the original construction plans or be engineered.

Sister or replace framing. For studs and similar repetitive framing members, it is often possible to place a new "sistered" member alongside the existing member and inter-fasten the two. This is often the quickest and least costly way to address damaged framing. This approach saves the extra work of having to remove a member and replace it. Removing and replacing members may also require shoring and damage may occur during the removal and replacement. Where this approach is not possible, such as for a header, removal and replacement of the damaged framing is appropriate.

Localized crushing on the order of 1/8 inch or less is not anticipated to affect the gravity or seismic performance of a house and is well within standard framing tolerance. Where crushing is more than 1/8 inch, shifting under gravity or subsequent lateral loading could occur. If there is an open gap in the framing, it is appropriate to take measures to close the gap. This could include installation of shims. Gaps can also be filled with either epoxy or a cementitious grout. Where access does not permit addressing gaps by one of these methods, framing member removal and replacement might be required.

4.5.11 Tie-Downs and Other Load Path Elements

This section addresses tie-downs and their anchorage, prescriptive load path elements and engineered load path elements.

Tie-Downs and Their Anchorage

Tie-down hardware and anchorage includes bracket- type tie-down devices and straps that are commonly used to provide anchorage between posts serving as shear wall boundary members and the foundation or a post in the story below. Also included are rod- or cable-type tie-down devices that are commonly used in stacked multi-story shear walls.

Tie-down hardware and anchorage repairs are to be determined based on the earthquake damage patterns provided in Table 4-25.

Table 4-25 Repair of Tie-Down Hardware and Anchorage

Earthquake Damage Pattern		*Repair Method*
4-25A	Local splitting of tie-down post or studs	No repair
4-25B	Extensive splitting of tie-down post or studs	Replace tie-down post or stud and reattach tie-down device
4-25C	Deformation of tie-down device	Remove and replace tie-down device
4-25D	Damage or fracture to tie-down anchor rods	Remove and replace rods at upper stories. Add new tie-down post and anchorage, as close to the existing as practical, if anchor rod to foundation is damaged
4-25E	Damage or fracture to fasteners between tie-down device and post or studs	Remove and replace fasteners if practical, otherwise replace tie-down device
4-25F	Spalling of concrete at tie-down anchor rods	Add new tie-down post and anchorage, as close to the existing as practical, if anchor rod to foundation is damaged
4-25G	Elongation or tearing of tie-down straps	Replace tie-down strap
4-25H	Withdrawal of nails or screws from tie-down straps	Replace tie-down strap
4-25I	Damage to through-rod system anchor plate at upper-floor level	Replace anchor plate
4-25J	Damage to through-rod system rods	Replace rod, replace other hardware as required

Note: Identify damage pattern and repair where damage has been identified during the damage investigation. Select damage pattern most representative of observed damage caused by or worsened by the earthquake. See Section 3.1.2 and Section 4.1.2 for discussion of worsened damage.

Prescriptive Load Path Elements

Many older houses were constructed using conventional construction (prescriptive construction) methods, and all houses have a good portion of their inter-member connections that are dictated by conventional construction practice. This includes the use of prescriptive fastening requirements from current and past building and residential codes and minimum housing standards. The fasteners dictated by prescriptive fastening requirements will form a portion of the load path for earthquake loading and are therefore subject to earthquake damage. Damage observed to similar fastening can be treated similarly.

Prescriptive load path repairs are to be determined based on the earthquake damage patterns provided in Table 4-26. For load path connections to foundation sill plates, see Section 4.4.

Table 4-26 Repair of Prescriptive Load Path Elements

Earthquake Damage Pattern		Repair Method
4-26A	Stud-to-sill plate end nailing showing visible indication that end nails have been partially or fully withdrawn and have not returned to their original position	Refasten using nails, screws, or framing clips
4-26B	Distress at ceiling joist, rafter, or roof blocking toenails to wall top plates	Refasten using nails, screws, or framing clips
4-26C	Distress at wall bottom plate to floor sheathing fasteners	Refasten using nails, screws, or framing clips
4-26D	Distress at wall bottom plate to rim joist or blocking fasteners	Refasten using nails, screws, or framing clips
4-26E	Distress at rim joist or blocking to top plate fasteners	Refasten using nails, screws, or framing clips
4-26F	Distress at floor joist-to-top plate fasteners	Refasten using nails, screws, or framing clips

Note: Identify damage pattern and repair where damage has been identified during the damage investigation. Select damage pattern most representative of observed damage caused by or worsened by the earthquake. See Section 3.1.2 and Section 4.1.2 for discussion of worsened damage.

The load path connections are most often repeated at very close intervals, such as nailing at 6-inches on center or fastening of joists or studs at 16-inches on center. The repetition of the fastening provides inherent redundancy. The natural variability of the framing being fastened makes it possible that occasional damage to connections might be seen without there being a pattern of systematic damage. This includes instances where either systematic damage occurs to connections or damage occurs to fastening of isolated framing members that are believed by the structural consultant to be key to the load path (e.g., major beams, posts).

Engineered Load Path Elements

Houses that have lateral-force-resisting systems (seismic and wind) designed using engineering methods will likely have additional load path connections that might be subject to damage. These are often pre-engineered, pre-manufactured connection hardware. One commonly found load path element is an L-shaped or flat shear clip that transfers shear as part of the earthquake load path. These are most often found connecting roof or floor framing to shear wall top plates or connecting shear wall bottom plates to floor framing or foundation sill plates below. Other commonly found elements, such as tie-downs, are used for uplift restraint to prevent walls from overturning due to in-plane shear.

Engineered load path repairs are to be determined based on the earthquake damage patterns provided in Table 4-27.

Table 4-27 Repair of Engineered Load Path Elements

Earthquake Damage Pattern		*Repair Method*
4-27A	Shear clip nails with limited indication of nail rocking, withdrawal of nails not exceeding 1/8 inch	No repair
4-27B	Shear clip nails with nail withdrawal exceeding 1/8 inch, limited deformation of the angle bracket	Reset nails in shear clip. Where nails cannot be reset, install new shear clip in the general vicinity
4-27C	Shear clip nail withdrawal greater than 1/4 inch, notable deformation of the shear clip, tearing of the shear clip	Add one new shear clip for each damaged shear clip. Existing shear clips can be abandoned in place if they do not interfere with placement of new shear clips or installation of sheathing or finish materials. Where interference occurs, remove existing shear clips
4-27D	Chord or collector blocking and straps with hairline splitting or withdrawal of nails not exceeding 1/8 inch	No repair
4-27E	Chord or collector blocking and straps with more extensive splitting or nail withdrawal	Abandon existing chord or collector connections in place where possible, and add new

Note: Identify damage pattern and repair where damage has been identified during the damage investigation. Select damage pattern most representative of observed damage caused by or worsened by the earthquake. See Section 3.1.2 and Section 4.1.2 for discussion of worsened damage.

The shear load path connections addressed in this section are most often repeated at very close intervals, which provides inherent redundancy. The natural variability of the framing being fastened makes it possible that occasional damage to connections might be seen without there being a pattern of systematic damage. This section addresses instances where systematic damage occurs to shear load path connections. Uplift load path connections can on occasion be redundant and treated similarly, but more often the uplift load path will be a key element and should be repaired even where there is no repetitive pattern of damage seen.

4.5.12 Gravity-Load-Carrying Elements

This section addresses damage to elements that carry gravity loads but are not addressed by other sections of Chapter 4. Included are posts that fall outside of framed walls. Repairs to gravity-load-carrying element are to be determined based on the earthquake damage patterns provided in Table 4-28.

Table 4-28 Repair of Gravity-Load-Carrying Elements

Earthquake Damage Pattern		*Repair Method*
4-28A	Post lean Not more than 1/4 inch per foot	No repair
4-28B	Post lean More than 1/4 inch per foot	Reset post where possible, replace post with new member otherwise
4-28C	Shifting of post at top not more than 1/4 inch relative to supported beam or other member	Secure post to supported beam or other member
4-28D	Shifting of post at top more than 1/4 inch relative to supported beam or other member	Return supported beam or other member to intended location to the extent possible and secure post to beam
4-28E	Shifting of post at bottom not more than 1/4 inch relative to original location	Secure post base to structure below
4-28F	Shifting of post at bottom more than 1/4 inch relative to original location	Return post bottom to intended location to the extent possible and secure post to structure below. Evaluate the effect of final post location on structure below

Note: Identify damage pattern and repair where damage has been identified during the damage investigation. Select damage pattern most representative of observed damage caused by or worsened by the earthquake. See Section 3.1.2 and Section 4.1.2 for discussion of worsened damage.

4.6 Other Seismic-Force-Resisting Elements

This section addresses alternatives to the site-built wood-frame wall bracing elements discussed in Section 4.5. This includes prescriptive bracing elements from current and past residential and building codes and engineered bracing elements. Alternative elements are most commonly used in a limited number of locations in houses otherwise braced by wood-frame walls.

4.6.1 Portal Frames

The construction of portal frames commonly used in walls surrounding garage doors and similar locations where bracing walls are limited to narrow wall piers is similar to that of engineered shear walls except that, in addition to tie-downs at the wall base, moment fixity is created between the top of the wall and a beam above, creating behavior similar to a moment frame. The moment connection is often achieved with either pairs of steel straps providing tension connections between each end of a wall pier and the beam above or with lapping of the wood structural panel sheathing over the face of the beam and a dense nail pattern at the lap. Portal frame provisions can be found in the prescriptive provisions of the IRC and IBC, but portal frames can also have an engineered design.

Most aspects of repairs for these shear walls are to be in accordance with Section 4.5.1 through Section 4.5.4. Repairs of aspects unique to portal frames are to be determined based on the earthquake damage patterns provided in Table 4-29.

Table 4-29 Repair of Portal Frames

Earthquake Damage Pattern		*Repair Method*
4-29A	Damage to sheathing or nailing at top-of-wall-pier moment connection to beam	Renail sheathing relocating nails where possible, otherwise replace and renail sheathing
4-29B	Damage to steel straps or their nailing at top-of-wall-pier moment connection to beam	Replace straps
4-29C	Limited splitting damage to beam at top-of-wall-pier moment connection to beam	Renail sheathing relocating nails
4-29D	Extensive splitting damage to beam at top-of-wall-pier moment connection to beam	Remove and replace beam
4-29E	Spalling damage to concrete curb at base	Remove damaged material and patch concrete
4-29F	Racking or leaning increased due to earthquake by more than 1/2 inch over the height of a story	Move the house as close to the original plumb position as practical. Repair or replace portal frame framing and sheathing as required
4-29G	Racking or leaning a total of more than 2 inches over the height of a story	Remove sheathing and finish materials, remove portal frame sheathing and framing, square up walls, refasten wall framing connections, and reinstall sheathing and finish materials

Note: Identify damage pattern and repair where damage has been identified during the damage investigation. Select damage pattern most representative of observed damage caused by or worsened by the earthquake. See Section 3.1.2 and Section 4.1.2 for discussion of worsened damage.

4.6.2 Proprietary Shear Walls

There are a large variety of pre-engineered, pre-manufactured shear walls that can be purchased and installed in houses. These include wood structural panel sheathed shear walls with proprietary designs and connectors, and shear walls using solid wood sections, steel sheet sections, steel truss elements, and wood truss elements. Because the construction of these is widely varied and the design is proprietary, repair of these elements will generally need to be developed in consultation with the element manufacturer. For this reason, the discussion of this section is limited to general areas that might experience damage without providing detail.

Repairs to proprietary shear walls are to be determined based on the earthquake damage patterns provided in Table 4-30.

Table 4-30 Repair of Proprietary Shear Walls

Earthquake Damage Pattern		Repair Method
4-30A	Damage to fastening of shear wall to structure above	Remove and replace fasteners
4-30B	Damage within proprietary shear wall	Consult with manufacturer
4-30C	Damage of anchorage of shear wall to foundation or supporting floor	Consult with manufacturer
4-30D	Racking or leaning increased due to earthquake by more than 1/2 inch over the height of a story	Move the house as close to the original plumb position as practical. Replace proprietary shear wall
4-30E	Racking or leaning a total of more than 2 inches over the height of a story	Remove sheathing and finish materials, remove proprietary shear wall, square up walls, replace proprietary shear wall, and reinstall sheathing and finish materials

Note: Identify damage pattern and repair where damage has been identified during the damage investigation. Select damage pattern most representative of observed damage caused by or worsened by the earthquake. See Section 3.1.2 and Section 4.1.2 for discussion of worsened damage.

4.6.3 Hillside Houses: Diagonal Braces

This section addresses diagonal braces often found on the downhill sides of hillside houses. Although the bracing elements discussed in this section contribute some strength and stiffness, the primary seismic bracing for hillside houses is typically the connection to the uphill foundation as discussed in Section 4.4.5.

Diagonal Lumber Bracing

In hillside houses, diagonal lumber bracing members used in an X-bracing or similar configuration to provide bracing between the house and the foundations. This includes one or more pairs of braces acting as seismic-force-resisting elements. Diagonal lumber sheathing is different from diagonal lumber bracing and is not addressed by this section (see Section 4.5.8).

Repairs to diagonal lumber bracing are to be determined based on the earthquake damage patterns provided in Table 4-31.

Table 4-31 Repair of Hillside House Diagonal Lumber Bracing

Earthquake Damage Pattern		Repair Method
4-31A	Hairline splitting of braces or framing to which the braces are attached	No repair
4-31B	Withdrawal of nails Damage to bolts or lag screws	Refasten braces
4-31C	More than hairline splitting or fracture of brace members or the framing members that braces are attached to (e.g., posts, beams)	Replace or reinforce split or fractured members

Note: Identify damage pattern and repair where damage has been identified during the damage investigation. Select damage pattern most representative of observed damage caused by or worsened by the earthquake. See Section 3.1.2 and Section 4.1.2 for discussion of worsened damage.

Diagonal Rod Bracing

In hillside houses, diagonal steel rod bracing includes members used in an X-bracing or similar configuration to provide bracing between the house and the foundations. Repairs to diagonal steel rod bracing are to be determined based on the earthquake damage patterns provided in Table 4-32.

Table 4-32 Repair of Hillside House Diagonal Rod Bracing

Earthquake Damage Pattern		*Repair Method*
4-32A	Limited deformation to rod-end connections or turn buckles	No repair
4-32B	Elongation of rods without fracture	Take practical measures to tighten rods, including tightening of turnbuckles or adjustment of end connections
4-32C	Fracture or failure of rod-end connections or turn buckles	Replace rods and connections
4-32D	Kinking, partial fracture, or full fracture of rods	Replace rods and, if damaged, connections
4-32E	More than hairline splitting of the framing members to which the braces are attached (e.g., posts, beams)	Reinforce or replace damaged framing members

Note: Identify damage pattern and repair where damage has been identified during the damage investigation. Select damage pattern most representative of observed damage caused by or worsened by the earthquake. See Section 3.1.2 and Section 4.1.2 for discussion of worsened damage.

4.6.4 Steel Moment Frames and Cantilevered Columns

Repairs to steel moment frames and steel cantilevered columns are to be determined based on the earthquake damage patterns provided in Table 4-33. Repairs to steel cantilevered columns are to be determined based on the earthquake damage patterns provided in Table 4-34.

Table 4-33 Repair of Steel Moment Frames

Earthquake Damage Pattern		*Repair Method*
4-33A	Bending of base plates localized between bolts and edge of plate	No repair
4-33B	Bending or partial fracture of base plate between bolts and moment frame column	Provide base plate augmentation
4-33C	Bending of anchor bolts without fracture	No repair
4-33D	Fracture of anchor bolts, spalling of concrete at anchor bolts, withdrawal or similar damage to anchor bolts	Provide base plate augmentation and new anchors
4-33E	Beam distortion without fracture	No repair

Table 4-33 Repair of Steel Moment Frames (continued)

Earthquake Damage Pattern		Repair Method
4-33F	Fracture at moment connection	Repair connection or replace frame
4-33G	Fastener or framing damage in load path connection to structure above	Repair and refasten load path connections
4-33H	Racking or leaning of less than 1/2 inch over a height of 8 feet	No repair
4-33I	Racking or leaning due to earthquake increased by more than 1/2 inch over the height of 8 feet	Straighten frame to the extent practical and refasten load path connections
4-33J	Racking or leaning more than 2 inches over the height of 8 feet	Remove and replace frame

Note: Identify damage pattern and repair where damage has been identified during the damage investigation. Select damage pattern most representative of observed damage caused by or worsened by the earthquake. See Section 3.1.2 and Section 4.1.2 for discussion of worsened damage.

Table 4-34 Repair of Steel Cantilevered Columns

Earthquake Damage Pattern		Repair Method
4-34A	Bending of base plates localized between bolts and edge of plate	No repair
4-34B	Bending or partial fracture of base plate between bolts and moment frame column	Provide base plate augmentation
4-34C	Bending of anchor bolts without fracture	No repair
4-34D	Fracture of anchor bolts, spalling of concrete at anchor bolts, withdrawal or similar damage to anchor bolts	Provide base plate augmentation and new anchors
4-34E	Distortion without fracture at base of column embedded in concrete foundation	No repair
4-34F	Fracture at base of column embedded in concrete foundation	Repair or replace column
4-34G	Fastener or framing damage in load path connection to structure above	Repair and refasten load path connections
4-34H	Racking or leaning of less than 1/2 inch over a height of 8 feet	No repair
4-34I	Racking or leaning due to earthquake increased by more than 1/2 inch over the height of 8 feet	Straighten column to the extent practical and refasten load path connections
4-34J	Racking or leaning more than 2 inches over the height of 8 feet	Straighten or remove and replace column

Note: Identify damage pattern and repair where damage has been identified during the damage investigation. Select damage pattern most representative of observed damage caused by or worsened by the earthquake. See Section 3.1.2 and Section 4.1.2 for discussion of worsened damage.

4.7 Floors, Ceilings, and Roofs

This section addresses floor, ceiling, and roof framing and diaphragms.

4.7.1 Floor, Ceiling, and Roof Framing

Floor, ceiling, and roof framing includes joists and rafters, supporting framing (e.g., beams, ledgers), and miscellaneous framing associated with the floor, ceiling, and roof systems. Repairs to floor, ceiling, and roof framing are to be determined based on the earthquake damage patterns provided in Table 4-35 and the more detailed descriptions that follow.

Table 4-35 Repair of Floor, Ceiling, and Roof Framing

Earthquake Damage Pattern		*Repair Method*
4-35A	Separation from supporting wall (ledger detached or similar condition)	Refasten or otherwise repair or augment in place
4-35B	Damage to joist hanger or fasteners	Repair in place when possible, remove and replace otherwise
4-35C	Splitting or fracture of framing	Repair in place by sistering or other strengthening when possible, otherwise remove and replace
4-35D	Loss of bearing: slight loss of bearing area, sliding less than 1/4 inch	Refasten in place
4-35E	Loss of bearing by more than 1/4 inch	Move back to intended location to the extent practical and secure in place
4-35F	Loss of bearing by more than 1 1/2 inches	Move back to intended location to the extent practical and secure in place or re-support in new location
4-35G	Out-of-plane distortion of more than 1/2 inch in 8 feet	Take practical measures to reduce out-of-plane distortion, including shimming at framing supports
4-35H	Out-of-plane distortion of more than 1 1/2 inches in 8 feet	Reduce out-of-plane distortion through shimming at supports, adding sister rafter or joists, or removing and reinstalling or replacing rafters or joists.

Note: Identify damage pattern and repair where damage has been identified during the damage investigation. Select damage pattern most representative of observed damage caused by or worsened by the earthquake. See Section 3.1.2 and Section 4.1.2 for discussion of worsened damage.

Re-support in new location. To the extent possible, loss of bearing at shifted members should be addressed by returning the member to the intended location and support. Where necessary a shifted member can be left at the new location, provided that an adequate load path can be provided in the shifted condition; the load path should be followed as far as required to assure adequate support of the loads in the new location.

Reduce out-of-plane distortion. Out-of-plane distortion of floor, ceiling, or roof framing most often occurs because of movement of the supporting walls or other supporting structure. Repairs to the supporting structure can often reduce the floor, ceiling, or wall distortion to an acceptable level.

4.7.2 Floor Diaphragms

Floor diaphragms can be lumber or wood structural panel sheathing fastened to solid sawn or engineered framing. Repairs to floor diaphragms are to be determined based on the earthquake damage patterns provided in Table 4-36.

Table 4-36 Repair of Floor Diaphragms

Earthquake Damage Pattern		*Repair Method*
4-36A	Nail pull-out, tear-out, or withdrawal	Refasten sheathing
4-36B	Hairline splitting of lumber sheathing	No repair
4-36C	More than hairline splitting of lumber sheathing	Relocate nails and refasten sheathing where possible, otherwise remove and replace sheathing
4-36D	Tearing or fracture of wood structural panel sheathing	Relocate nails and refasten sheathing where possible, otherwise remove and replace sheathing
4-36E	Detachment or buckling of sheathing	Relocate nails and refasten sheathing where possible, otherwise remove and replace sheathing
4-36F	In-plane racking or distortion not more than 1/2 inch in 8 feet	No repair to diaphragm, repair finish materials as required
4-36G	In-plane racking or distortion more than 1/2 inch in 8 feet	Remove distortion to the extent possible, refasten sheathing if required
4-36H	In-plane racking or distortion more than 2 inches in 8 feet	Remove sheathing and finish materials, square up diaphragm and reinstall sheathing and finish materials

Note: Identify damage pattern and repair where damage has been identified during the damage investigation. Select damage pattern most representative of observed damage caused by or worsened by the earthquake. See Section 3.1.2 and Section 4.1.2 for discussion of worsened damage.

4.7.3 Ceiling Diaphragms

Ceiling diaphragms include ceilings sheathed with gypsum wallboard, plaster on gypsum, or wood lath that contribute strength and stiffness to resist seismic loading. Section 4.7.3 addresses the ceiling finish material while Section 4.7.1 addresses the supporting framing. Repairs to gypboard ceiling diaphragms are to be determined based on the earthquake damage patterns provided in Table 4-37 and the more detailed descriptions that follow. Repairs to plaster on wood lath ceiling diaphragms are to be determined based on the earthquake damage patterns provided in Table 4-38 and the more detailed descriptions that follow. Repairs to plaster on gypsum lath ceiling diaphragms are to be determined based on the earthquake damage patterns provided in Table 4-39 and the more detailed descriptions that follow.

Table 4-37 Repair of Gypboard Ceiling Diaphragms

Earthquake Damage Pattern		Repair Method
4-37A	Short cracks up to 1/64 inch	No repair
4-37B	Cracks following taped joints or corner bead, puckering or buckling of tape at joint	Re-tape joints as required, rout, patch, and refinish
4-37C	Nail pops	Add drywall screw 1 inch from original fastener, set or remove original fastener, patch and refinish
4-37D	Cracks up to 1/8-inch wide through the board	Remove and replace gypboard to nearest joists beyond crack (minimum 32 inches by 48 inches), refinish
4-37E	Crushing and puckering of gypboard at panel edges	Remove and replace gypboard to nearest joists beyond damage (minimum 32 inches by 48 inches), refinish
4-37F	Tear-out of fasteners at panel edges	Remove and replace gypboard to nearest joists beyond damage (minimum 32 inches by 48 inches), refinish
4-37G	In-plane racking or distortion not more than 1/2 inch in 8 feet	No repair to diaphragm, repair finish materials as required
4-37H	In-plane racking or distortion more than 1/2 inch in 8 feet	Remove distortion to the extent possible, refasten sheathing if required
4-37I	In-plane racking or distortion more than 2 inches in 8 feet	Remove sheathing and finish materials, square up diaphragm framing and reinstall materials
4-37J	Back face of board is slotted by fastener to maximum length of 1/2 inch	Add drywall screw 2 inches from original fastener, set or remove original fastener, patch and refinish
4-37K	Back face of board is slotted by fastener to length greater than 1/2 inch	Remove and replace gypboard to nearest joists beyond damage (minimum 32 inches by 48 inches), refinish. Damage may extend for full ceiling plane
4-37L	Gypboard is detached from ceiling framing	Remove and replace entire board, tape and finish

Notes: [a] Damage patterns with gray shading are also identified in the *General Guidelines*.
[b] Identify damage pattern and repair where damage has been identified during the damage investigation. Select damage pattern most representative of observed damage caused by or worsened by the earthquake. See Section 3.1.2 and Section 4.1.2 for discussion of worsened damage.

Where cracking follows panel joints or corner beads, existing tape and compound should be removed. The joint should then be retaped, retextured, and repainted. When determining the area to be retextured and repainted, consideration should be given to obtaining a reasonably uniform appearance.

Short (less than about 6-inches long) cracks less than about 1/64-inch wide and extending from ceiling corners may be patched using drywall tape and joint compound, and then retextured and repainted. When determining the area to be retextured and repainted, consideration should be given to obtaining a reasonably uniform appearance.

Where cracks greater than about 6-inches long extend through the thickness of the drywall, the cracked piece should be removed to the nearest joist on either side of the crack (32-inch minimum width of removed gypboard) and replaced, retextured, and repainted.

Nail pops may be repaired by adding a drywall screw adjacent to the nail pop, resetting or removing the popped fastener, patching, retexturing, and repainting. When determining the area to be retextured and repainted, consideration should be given to obtaining a reasonably uniform appearance.

Fractured gypsum wallboard panels should be replaced in kind. Where the attachment of the drywall to the framing has loosened significantly, new fasteners should be installed around the wall perimeter and along the panel joints showing signs of relative movement. The repair areas should be refinished as necessary to match adjacent areas. When determining the area to be retextured and repainted, consideration should be given to obtaining a reasonably uniform appearance.

Table 4-38 Repair of Plaster on Wood Lath Ceiling Diaphragms

Earthquake Damage Pattern		*Repair Method*
4-38A	Limited cracks up to 1/64-inch wide	Patch and refinish
4-38B	Limited cracks up to 1/8-inch wide	Rout, patch, and refinish
4-38C	Local compression buckling of finish coat	Remove plaster locally at spall, route, patch, and refinish
4-38D	Extensive cracking but plaster is still tight to lath	Remove and reinstall plaster and refinish
4-38E	In-plane racking or distortion not more than 1/2 inch in 8 feet	No repair to diaphragm, repair finish materials as required
4-38F	In-plane racking or distortion more than 1/2 inch in 8 feet	Remove distortion to the extent possible, refasten sheathing if required
4-38G	In-plane racking or distortion more than 2 inches in 8 feet	Remove sheathing and finish materials, square up diaphragm and reinstall materials
4-38H	Plaster spalling	Remove plaster and wood lath, install wood structural panel sheathing, and install gypsum wallboard

Notes: [a] Damage patterns with gray shading are also identified in the *General Guidelines*.
[b] Identify damage pattern and repair where damage has been identified during the damage investigation. Select damage pattern most representative of observed damage caused by or worsened by the earthquake. See Section 3.1.2 and Section 4.1.2 for discussion of worsened damage.

Replace with gypboard over wood structural panel sheathing. This repair is intended to restore strength and at the same time provide the same total thickness as the plaster and wood lath that was removed. This allows existing trim to be reused without having to make potentially costly adjustments. The total thickness of plaster and wood lath can vary but is commonly about 7/8 inch. Restoring this thickness will commonly require use of 3/8-inch plywood and 1/2-inch gypboard panels. The actual existing material thickness will need to be determined when repairs are implemented, and the thickness of the plywood sheathing adjusted accordingly. Plywood sheathing should have all edges blocked and be nailed with not less than 8-penny common nails spaced not more than 4-inches on center at panel edges and 12-inches on center at other supports.

Table 4-39 Repair of Plaster on Gypsum Lath Ceiling Diaphragms

Earthquake Damage Pattern		*Repair Method*
4-39A	Limited cracks up to 1/64-inch wide	Patch and refinish
4-39B	Limited cracks up to 1/8-inch wide	Rout, patch, and refinish
4-39C	Local compression buckling of finish coat	Remove plaster locally at spall, route, patch, and refinish
4-39D	Extensive cracking but plaster is still tight to lath	Remove and reinstall plaster and refinish
4-39E	In-plane racking or distortion not more than 1/2 inch in 8 feet	No repair to diaphragm, repair finish materials as required
4-39F	In-plane racking or distortion more than 1/2 inch in 8 feet	Remove distortion to the extent possible, refasten sheathing if required
4-39G	In-plane racking or distortion more than 2 inches in 8 feet	Remove sheathing and finish materials, square up diaphragm and reinstall materials
4-39H	Plaster spalling	Remove plaster and gypsum lath and replace with gypsum wallboard over a gypsum wallboard or wood structural panel sheathing base as required to match existing thickness

Notes: [a] Damage patterns with gray shading are also identified in the *General Guidelines*.

[b] Identify damage pattern and repair where damage has been identified during the damage investigation. Select damage pattern most representative of observed damage caused by or worsened by the earthquake. See Section 3.1.2 and Section 4.1.2 for discussion of worsened damage.

Where the plaster is cracked but remains firmly attached to the gypsum lath, repairs can be accomplished by cleaning the crack, and patching, texturing, and painting to match the existing surface texture and finish. When determining the area to be retextured and repainted, consideration should be given to obtaining a reasonably uniform appearance.

Where the lath has fractures (or where plaster damage suggests fracture of the gypsum lath), the damaged pieces should be removed to soundly attached plaster or lath or both, new lath installed, and the area replastered. Where large areas of repair are involved and it is more economical to do so, plaster and lath should be removed to the limits of the panel and replaced with gypsum wallboard. When determining the area to be refinished, consideration should be given to obtaining a reasonably uniform appearance.

4.7.4 Roof Diaphragms

Repairs to roof diaphragms are to be determined based on the earthquake damage patterns provided in Table 4-40.

Table 4-40 Repair of Roof Diaphragms

Earthquake Damage Pattern		Repair Method
4-40A	Nail pull-out, tear-out, or withdrawal	Refasten sheathing
4-40B	Hairline splitting of lumber sheathing	No repair
4-40C	More than hairline splitting of lumber sheathing	Remove and replace lumber sheathing
4-40D	Tearing or fracture of wood structural panel sheathing	Remove and replace sheathing
4-40E	Detachment or buckling	Remove and replace sheathing
4-40F	In-plane racking or distortion not more than 1/2 inch in 8 feet	No repair to diaphragm, repair finish materials as required
4-40G	In-plane racking or distortion more than 1/2 inch in 8 feet	Remove distortion to the extent possible, refasten sheathing if required
4-40H	In-plane racking or distortion more than 2 inches in 8 feet	Remove sheathing and finish materials, square up diaphragm and reinstall materials

Note: Identify damage pattern and repair where damage has been identified during the damage investigation. Select damage pattern most representative of observed damage caused by or worsened by the earthquake. See Section 3.1.2 and Section 4.1.2 for discussion of worsened damage.

4.8 Fireplaces and Chimneys

This section addresses damage to brick masonry chimneys and fireboxes, their anchor straps to the house, braces to the house roof, and anchored masonry veneer fireplace surrounds.

4.8.1 Chimneys and Fireboxes

Brick masonry chimneys might or might not have embedded reinforcing steel and might or might not be strapped to the supporting house at each floor, ceiling, and roof level. The majority of existing house chimneys are anticipated to be unreinforced and either not strapped or not adequately strapped.

Repairs to brick masonry chimneys and fire boxes are to be determined based on the earthquake damage patterns provided in Table 4-41. Repairs to anchorage straps between the chimney and house floor, ceiling, and roof framing are to be determined based on the earthquake damage patterns provided in Table 4-42. Repairs to braces between the chimney and the roof are to be determined based on the earthquake damage patterns provided in Table 4-43. The items discussed only address what is visible to the structural consultant at the house site without removal of materials or specialty inspection equipment. Additional repair scope might be identified by a chimney inspection specialist.

Table 4-41 Repair of Chimneys and Fireboxes

Earthquake Damage Pattern		Repair Method
4-41A	Foundation settlement relative to the house with firebox or chimney lean not more than 1/2 inch over a height of 8 feet	Refasten framing, straps, and trim, if exist, to house
4-41B	Foundation settlement relative to the house with firebox or chimney lean more than 1/2 inch over a height of 8 feet	Take practical measures to relevel the chimney foundation and reduce chimney lean. Refasten framing and straps, if exist, to house
4-41C	Foundation settlement relative to the house with firebox or chimney lean more than 2 inches over a height of 8 feet	Remove and reconstruct chimney with factory-built fireplace and light-frame walls
4-41D	Cracks in individual bricks, less than 1/16-inch wide, no offset, slope or settlement	No repair
4-41E	Crack in bricks, less than 1/8-inch wide, no offset or spall	Remove and replace bricks
4-41F	Spall in chimney bricks or mortar, extending only partially through width of bricks	Remove and replace bricks
4-41G	Crack in mortar joint, less than 1/8-inch wide, no offset, slope, or settlement	Repoint mortar joints
4-41H	Crack in brick or mortar joint in chimney above roof line, more than 1/8-inch wide, with or without shifting of the chimney	Cap chimney just above roof line if loss of chimney use is acceptable. Reconstruct chimney from the top of fire box up with listed metal flue and light-frame walls if continued use of chimney is intended
4-41I	Crack in brick or mortar joint in interior chimney above ceiling line, more than 1/8-inch wide, with or without shifting of the chimney	Cap chimney just above ceiling line if loss of chimney use is acceptable. Reconstruct chimney from the top of fire box up with listed metal flue and light-frame walls if continued use of chimney is intended
4-41J	Crack in brick or mortar joint in chimney above top of fire box, more than 1/8-inch wide, loss of use of chimney is acceptable	Reconstruct chimney from the top of fire box up with listed metal flue and light-frame walls, or reconstruct chimney from foundation up with factory-built fireplace and light-frame walls

Notes: [a] Damage patterns with gray shading are also identified in the *General Guidelines*.
[b] Identify damage pattern and repair where damage has been identified during the damage investigation. Select damage pattern most representative of observed damage caused by or worsened by the earthquake. See Section 3.1.2 and Section 4.1.2 for discussion of worsened damage.

Practical measures to relevel the firebox foundation might include jacking an edge of the firebox foundation and re-supporting it on a new grout or lean-concrete spread footing cast under the existing footing.

Guidance for partial and full reconstruction of fireplaces and chimneys can be found in FEMA DR-4193-RA1 (FEMA, 2015) and FEMA P-1100 (FEMA, 2018).

Table 4-42 Repair of Chimney Anchor Straps to House

Earthquake Damage Pattern		Repair Method
4-42A	Damage to steel straps anchoring chimney to floor, ceiling, or roof framing	Augment or replace existing strap
4-42B	Damage to floor, ceiling, of roof framing	Sister existing framing where possible, otherwise replace
4-42C	Damage to fasteners between steel strap and floor, ceiling, or roof framing	Replace existing fasteners
4-42D	Damage to anchorage of steel straps into masonry chimney	Provide replacement strap external to chimney

Note: Identify damage pattern and repair where damage has been identified during the damage investigation. Select damage pattern most representative of observed damage caused by or worsened by the earthquake. See Section 3.1.2 and Section 4.1.2 for discussion of worsened damage.

Provide replacement strap. Most chimney straps installed as part of original construction are embedded in the masonry. It is seldom practical for replacement anchors to be similarly embedded or adequately fastened to the chimney. For this reason, replacement straps generally wrap around the chimney exterior and do not necessarily need to be anchored to the chimney masonry. Caution should be exercised when anchoring to an existing masonry chimney, as anchor installation can create a fire hazard by cracking the surrounding masonry or piercing the fireplace flue.

Table 4-43 Repair of Chimney Bracing to Roof

Earthquake Damage Pattern		Repair Method
4-43A	Damage to bracing members between chimney and roof	Sister bracing members where possible, otherwise replace
4-43B	Damage to attachment of bracing members to roof	Repair anchorage where possible, otherwise replace
4-43C	Damage to attachment of bracing members to chimney	Repair anchorage where possible, otherwise replace bracing members

Note: Identify damage pattern and repair where damage has been identified during the damage investigation. Select damage pattern most representative of observed damage caused by or worsened by the earthquake. See Section 3.1.2 and Section 4.1.2 for discussion of worsened damage.

Care should be taken in the damage investigation of chimneys with diagonal braces to the roof, as these braces have been associated with earthquake-induced damage to the chimney at or near the roof line.

4.8.2 Interior Fireplace Surrounds

Repairs to anchored veneer fireplace surrounds are to be determined based on the earthquake damage patterns provided in Table 4-44 and the more detailed descriptions that follow. Adhered veneer is considered a wall finish material that does not provide significant bracing strength or stiffness and should be addressed in accordance with the *General Guidelines*.

Table 4-44 Repair of Interior Anchored Veneer Fireplace Surrounds

Earthquake Damage Pattern		Repair Method
4-44A	Veneer has detached and moved away from the wall not more than 1/2 inch	Refasten veneer to wall framing using retrofit anchors
4-44B	Veneer has detached and moved away from the wall more than 1/2 inch	Remove and reinstall veneer in accordance with applicable building or residential code
4-44C	Veneer has collapsed away from the wall	Remove and reinstall veneer in accordance with applicable building or residential code

Note: Identify damage pattern and repair where damage has been identified during the damage investigation. Select damage pattern most representative of observed damage caused by or worsened by the earthquake. See Section 3.1.2 and Section 4.1.2 for discussion of worsened damage.

Refasten veneer with retrofit anchors. Retrofit anchors are available that can be installed through holes drilled in the veneer mortar joints. Installing these anchors requires that the studs in the wall behind be accurately located to ensure that new anchors are well fastened to studs. Where locating the studs will be difficult, it may be less costly to remove and reinstall the veneer.

Reinstall veneer. When veneer is reinstalled, installation should be in accordance with the current building or residential code, including requirements for anchor type and spacing and for wire reinforcing in the mortar joints.

4.9 Nonstructural Components

This section addresses damage to bracing and anchorage of nonstructural components as discussed in Section 3.9. Where assessment and repair of equipment (as opposed to bracing or anchorage) is required, this should be performed by a technical consultant specializing in the appropriate type of system.

Repairs to component bracing and anchorage are to be determined based on the earthquake damage patterns provided in Table 4-45.

Table 4-45 Repair of Nonstructural Component Bracing and Anchorage

Earthquake Damage Pattern		Repair Method
4-45A	Loose component connection or anchorage	Repair or replace connection or anchorage
4-45B	Failed component connection or anchorage	Replace connection or anchorage
4-45C	Bent or damaged component bracing or support	Repair or replace bracing or support
4-45D	Fractured or failed component bracing or support	Replace bracing or support

Note: Identify damage pattern and repair where damage has been identified during the damage investigation. Select damage pattern most representative of observed damage caused by or worsened by the earthquake. See Section 3.1.2 and Section 4.1.2 for discussion of worsened damage.

4.10 Appurtenances

Appurtenances include covered porches and entryways, attached decks, attached carports or patio covers, and exterior stairs, and their supporting walls, posts, and connections. The emphasis of this section is on appurtenances that are attached to and rely on the house's main structure for gravity support or for lateral resistance to ground shaking. See Section 3.10 for further discussion. Included is the structure of the appurtenance and the connection or anchorage of the appurtenance to the house.

4.10.1 Appurtenances

This section discusses damage to the structure of the appurtenance. Repairs to appurtenances are to be determined based on the earthquake damage patterns provided in Table 4-46.

Table 4-46 Repair of Appurtenances

Earthquake Damage Pattern		*Repair Method*
4-46A	Damage to framing member	Repair in place by sistering or similar where possible, otherwise replace
4-46B	Loss of bearing for framing member	Reestablish bearing and fasten in place
4-46C	Damage to supporting post or wall pier	Repair in place where possible, otherwise replace
4-46D	Damage to bracing elements internal to the appurtenance	Repair or refasten in place where possible, otherwise replace

Note: Identify damage pattern and repair where damage has been identified during the damage investigation. Select damage pattern most representative of observed damage caused by or worsened by the earthquake. See Section 3.1.2 and Section 4.1.2 for discussion of worsened damage.

4.10.2 Connections of Appurtenances to the House

This section discusses damage to the connections between an appurtenance and the main structure of the house. Repairs to connections of appurtenances to the house are to be determined based on the earthquake damage patterns provided in Table 4-47.

Table 4-47 Repair of Connections of Appurtenances to the House

Earthquake Damage Pattern		*Repair Method*
4-47A	Damage to ledger or similar member	Repair in place where possible, otherwise replace
4-47B	Damaged to fastening of ledger or similar member to the house	Repair in place where possible, otherwise replace
4-47C	Damage to fastening of ledger or similar member to the appurtenance	Repair in place where possible, otherwise replace

Note: Identify damage pattern and repair where damage has been identified during the damage investigation. Select damage pattern most representative of observed damage caused by or worsened by the earthquake. See Section 3.1.2 and Section 4.1.2 for discussion of worsened damage.

CHAPTER 5

Background Information: Geotechnical

5.1 Introduction

This chapter provides background material relevant to a geotechnical consultant's assessment of earthquake-induced permanent ground deformation. It is intended to complement Chapter 6, which presents a description of geotechnical post-earthquake site investigation and methods for damage identification, and Chapter 7, which presents methods for determining if permanent ground deformation occurred and estimating the resulting ground displacements. Chapter 7 also describes repair and mitigation strategies.

During earthquakes, buildings and other surface improvements can be damaged directly by strong shaking or from earthquake-induced permanent displacements of the ground. For the purpose of this document, *earthquake-induced permanent ground deformation* is defined as any earthquake-generated process that leads to deformations within a soil medium, which in turn results in permanent horizontal or vertical displacement of the ground surface. The following mechanisms of earthquake-induced permanent ground deformation have been documented in past earthquakes:

- surface fault rupture
- liquefaction
- seismic compression
- landsliding
- retaining wall deformation and associated retained ground deformation

Table 5-1 summarizes the earthquake-induced permanent ground deformation mechanisms discussed in this document, and Section 5.2 through Section 5.6 present background information for each mechanism.

When considering if these mechanisms of permanent ground deformation affected a site, it is important to recognize the requisite conditions for their occurrence. For example, the requisite condition for surface fault rupture is simply proximity of the site to the ruptured fault. Ground displacements are naturally greatest at sites located directly over the ruptured fault, but significant secondary deformations can also occur away from the main rupture. Soil liquefaction requires the presence of ground water, soil materials susceptible to liquefaction, and dynamic loading of sufficient amplitude and duration to trigger liquefaction in those materials. Seismic compression requires relatively strong shaking and unsaturated soil. The requisite conditions for landsliding are the presence of sloping ground and the presence of combined static and dynamic shear stresses that exceed material strengths.

Table 5-1 Summary of Earthquake-Induced Permanent Ground Deformation Mechanisms

Ground Deformation Mechanism	*Description*
Surface Fault Rupture (Section 5.2)	Fault rupture involves relative displacements (i.e., *slip*) of blocks of rock on opposite sides of the fault surface. There are two types of ground displacement resulting from faulting: principal faulting and distributed faulting. • *Principal faulting* is slip along the main plane (or planes) responsible for the release of seismic energy during the earthquake. • *Distributed faulting* is displacement that occurs on discontinuities, such as other faults, shears, or fractures in the vicinity of the principal rupture in response to the principal faulting. The term distributed faulting can also involve ground warping that does not involve distinct displacements across discontinuities.
Liquefaction (Section 5.3)	Liquefaction is defined as the transformation of a granular soil from a solid state to a liquefied state as a consequence of increased pore pressure and reduced effective stress. The generation of excess pore pressure under undrained loading conditions leads to a loss of soil strength and stability, resulting in permanent ground deformation. Residential properties have typically been subject to earthquake damage due to ground deformations from level-ground liquefaction and lateral spreading.
Seismic Compression (Section 5.4)	Seismic compression is defined as the accrual of contractive volumetric strains in unsaturated soil during strong shaking from earthquakes. Characteristic fill deformation features include cracks at cut or fill contacts due to differential settlement, ground cracks due to differential settlement across the surface of fill pads, and ground cracks due to lateral extension of fill pads towards the slope face. The requisite conditions for seismic compression are simply the presence of unsaturated soil and large amplitude earthquake ground motions.
Landsliding (Section 5.5)	Earthquake-induced landslides involve permanent shear deformations within geologic media. Landslides can be subdivided into several generalized categories. • Masses of disrupted slide material, such as rock falls or avalanches. • Relatively coherent slide masses whose displacement is accommodated along well-defined slip surfaces or across relatively broad, distributed shear zones. • Lateral spreads and flows associated with soil strength loss due to pore pressure increase.
Retaining Wall Deformation (Section 5.6)	Retaining wall failure is defined as excessive permanent deformation of the retaining structure. Deformations may consist of sliding, rotation, bending, or settlement, and manifestation of these mechanisms will depend on the type of retaining structure. Retaining structure movement during an earthquake depends upon the behavior of the soil beneath the wall, response of the backfill behind the wall, structural response of the wall itself, and the nature of the earthquake ground motions.

If all sites were pristine and stable in the absence of earthquakes, identification of earthquake-induced permanent ground deformation during a post-earthquake site investigation would be straightforward. However, a number of non-seismic geotechnical processes can also result in ground displacement that likewise may damage structures and surface improvements. These are discussed in Section 5.7 and include:

- consolidation settlement
- immediate settlement

- hydro-compression settlement
- expansive soil movement
- slope creep
- non-seismic landsliding
- non-seismic retaining wall deformation and failure

During a post-earthquake investigation of a flat site, care should be taken to distinguish ground settlements or heave that are characteristic of static volume change phenomena—such as from expansive soil—from ground settlements associated with liquefaction or seismic compression. Likewise, post-earthquake investigation of sloping sites should distinguish long-term slope instability (i.e., landslides), creep, or retaining wall movements from ground deformations associated with earthquake-induced landsliding.

Lastly, a hybrid failure mode, which is a combination of non-seismic soil movement and seismic shaking of improvements, is often misidentified as earthquake-induced soil movement. This mode of damage occurs when improvements on the surface (typically nonstructural concrete slabs-on-grade) span over and mask underlying soil movement (typically settlement of fill within or adjacent to the footprint of a building). With little or no loading on the slab, the unsupported condition can be marginally stable for many years. However, even small earthquake-induced displacements or forces may be sufficient to cause failure of the slab, which drops onto the previously settled underlying soil.

Chapter 5 through Chapter 7 provide general guidelines for post-earthquake investigation to evaluate the potential occurrence of earthquake-induced permanent ground deformation phenomena and to develop conceptual repair and mitigation strategies. The guidance is intended to be consistent with the current literature, research, and engineering standards and codes.

Frank Lloyd Wright and a Hybrid Failure Mode

A classic example of a hybrid failure mode was the floor slab of Frank Lloyd Wright's Hanna House on the Stanford University campus in California during the 1989 Loma Prieta earthquake. During construction of the house in the 1930s, poorly compacted clay fill had been placed within the perimeter foundation to support the concrete floor slab. Over time, that fill settled under its own weight, creating a void beneath the slab and leaving the nonstructural floor slab largely unsupported except on the edges, where it rested on the wall foundations. Shaking caused sagging and cracking of the floor slab. While the soil at the site had been unaffected by the earthquake, a casual inspection of the damaged floor slab could easily lead one to the erroneous conclusion that earthquake shaking had caused the soil to settle.

5.2 Surface Fault Rupture

Earthquakes result from sudden slip across a fault surface. Earthquakes on faults are generated in rock deep within the earth's crust, with typical focal depths (i.e., the depth at which slip originates) in California being on the order of about 3 miles to 12 miles (5 km to 20 km). Fault rupture involves relative displacements (i.e., slip) of blocks of rock on opposite sides of the fault surface. The slip of a fault during an earthquake results in large-scale relative displacements of the earth on opposite sides of the fault. These relative displacements can be as large as about 32 feet (10 m) in the case of large magnitude earthquakes.

When fault slip extends to the ground surface, the resulting ground displacements are termed “surface fault rupture” (Figure 5-1). Examples from California with surface fault rupture include the 1906 San Francisco, 1971 San Fernando, 1992 Landers, and 1999 Hector Mine earthquakes.

Figure 5-1 Surface fault rupture from the 1971 San Fernando, California earthquake and associated damage to pavement and structures in the Sylmar area (photo credit: National Information Service for Earthquake Engineering, or NISEE).

The direction of relative displacements of blocks of rock on opposite sides of the fault surface dictates the rupture mechanism assigned to an earthquake on the fault. Two general categories of earthquakes are strike-slip and dip-slip, although earthquakes can occur as combinations of the two in which case they are referred to as oblique. Figure 5-2 illustrates strike-slip and dip-slip faults. Dip-slip faults are represented by the reverse and normal mechanisms, which are defined below.

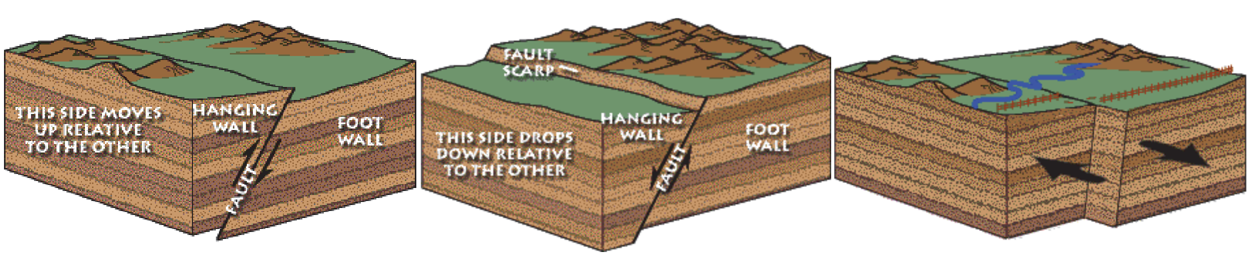


Figure 5-2 Schematic illustrations of fault rupture mechanisms (image credit: J. P. Stewart).

The term strike refers to the orientation of a line formed by the intersection of the fault plane with a horizontal plane. The orientation of the line is described by its azimuth (e.g., North-30 degrees-West). Strike-slip earthquakes involve fault slip parallel to the strike. The ground displacement that occurs during strike slip earthquakes is predominantly horizontal and in the general direction of the fault strike. These types of earthquakes can be further described as right-lateral or left-lateral strike slip. These terms refer the direction of horizontal displacement as an observer looks across the fault. In California, there are numerous major faults that produce right-lateral strike-slip earthquakes, including the San Andreas, Hayward, and Newport-Inglewood faults.

The term dip refers to the vertical angle between the fault plane and a horizontal line drawn perpendicular to the strike. The two sides of the fault are referred to as the hanging wall (i.e., the block of earth overlying the dipping fault) and the foot wall (i.e., the block of earth below the dipping fault). Dip-slip earthquakes involve fault slip parallel to the dip. The ground displacement that occurs during dip-slip earthquakes has vertical and horizontal components assuming the dip is not vertical. Dip slip earthquakes can be further described as normal or reverse (or thrust) earthquakes, and the faults producing such earthquakes are referred to as normal faults or reverse/thrust faults. Reverse faults are common in southern California and were responsible for the 1971 San Fernando and 1994 Northridge earthquakes. Normal faults occur in portions of eastern California, including the Long Valley area.

It is important to distinguish between two types of ground displacement from faulting (i.e., two types of surface fault rupture): principal faulting and distributed faulting. Principal faulting is slip along the main plane (or planes) responsible for the release of seismic energy during the earthquake (Youngs et al., 2003). When the principal fault rupture extends to the surface, it may be manifest along a single narrow trace or over a zone that may be several meters in width. The requisite condition for principal faulting to occur at a given site is direct proximity of the site to the fault that produced the earthquake (i.e., the fault that was the primary source of energy release for the earthquake). Sometimes earthquakes do not produce principal surface fault rupture, in which case they are referred to as "blind," an example of which is the 1994 Northridge earthquake.

Distributed faulting is displacement that occurs on discontinuities, such as other faults, shears, or fractures, in the vicinity of the principal rupture in response to the principal faulting (Youngs et al., 2003). The term distributed faulting can also involve ground warping that does not involve distinct displacements across discontinuities. Distributed faulting is discontinuous in nature and occurs over a zone that can, for certain types of faulting, extend up to several kilometers from the principal rupture. The requisite condition for distributed faulting is proximity to the fault that produced the earthquake, although "proximate" distances in this case can be much larger (on the order of hundreds of meters to kilometers) than in the case of principal faulting (on the order of meters to tens of meters). For a given property located off of the main fault trace, the probability of distributed rupture is generally fairly low. In Chapter 7, procedures are presented for evaluating the probability of distributed surface rupture.

For strike-slip earthquakes, principal faulting occurs along relatively straight and narrow segments whose endpoints are defined by the end of the rupture or step-overs to other segments. Within the step-over zones, broad zones of distributed normal or reverse faulting can occur due to the extension or compression within these zones. An example is shown in Figure 5-3, where normal faulting between strike-slip fault segments caused down-dropping of a coastal region in Turkey that led to flooding. For dip-slip earthquakes, principal faulting can occur along irregular alignments, although the shape of the rupture surface may be predictable based on surface topography and geologic mapping. As shown in Chapter 7, distributed faulting from dip-slip earthquakes is more pronounced on the hanging wall than on the foot wall.

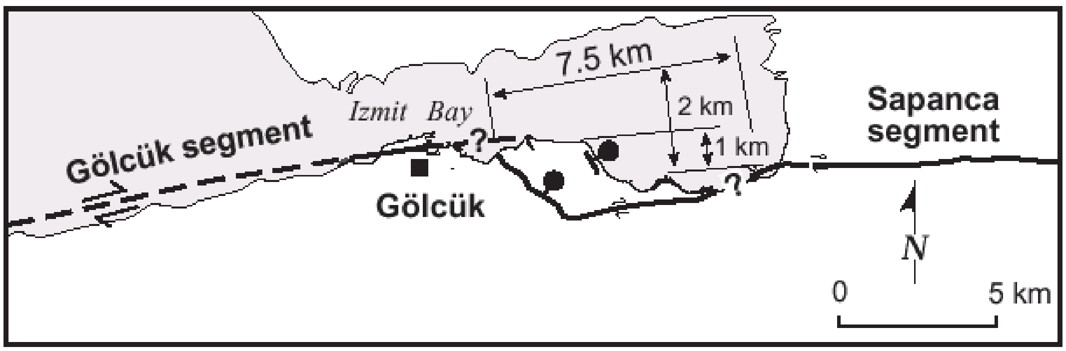


Figure 5-3 Example of step-over in ruptured fault in the 1999 Kocaeli, Turkey earthquake. Step-over occurred between the Golcuk and Sapanca segments, causing extension in coastal areas of Golcuk, as indicated by closed circles (Lettis et al., 2000).

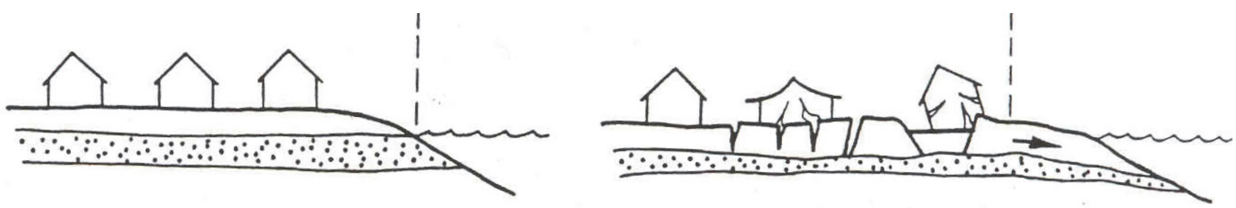
5.3 Liquefaction

It has long been recognized that loose, dry, granular soils densify when sheared slowly under drained conditions. If the soils are saturated (i.e., undrained conditions) and shearing occurs rapidly, the contractive nature of the soil results in an increase in pore water pressure and an associated decrease in effective stress and shear strength. This process is known as liquefaction. Liquefaction is defined as the transformation of a granular soil from a solid state to a liquefied state as a consequence of increased pore pressure and reduced effective stress (Committee on Soil Dynamics of the Geotechnical Engineering Division, 1978). Field case histories and liquefaction assessment procedures have focused primarily on sandy soils; however, recent case histories have demonstrated the devastating effects of liquefaction in silty soils (e.g., 1999 Adapazari, Turkey) and gravelly soils (e.g., 2016 Wellington, New Zealand). Herein, the initiating disturbance for liquefaction is assumed to be cyclic shear deformations resulting from an earthquake.

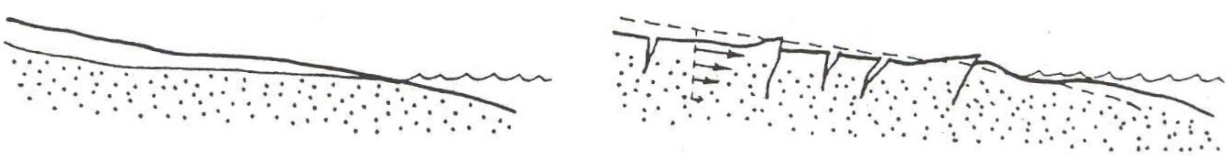
Soil softening and loss of shear strength from liquefaction allows large cyclic and perhaps permanent ground deformations to occur, both of which can be damaging to structures and surface improvements. The loss of shear strength associated with liquefaction can create ground deformations and instability (Figure 5-4 and Figure 5-5). For example, post-liquefaction dissipation of pore pressures leads to volumetric strains, which may cause ground settlement and lateral deformations in sloping ground.

Moreover, loss of soil shear strength can lead to instability if static shear stresses are present in the ground. To provide a framework for discussing the occurrence of soil deformations due to excess pore pressure generation under undrained loading conditions (i.e., "liquefaction"), the phenomena are grouped into the general categories of flow failure and cyclic mobility, where level-ground liquefaction is a special case of cyclic mobility (Kramer, 1996).

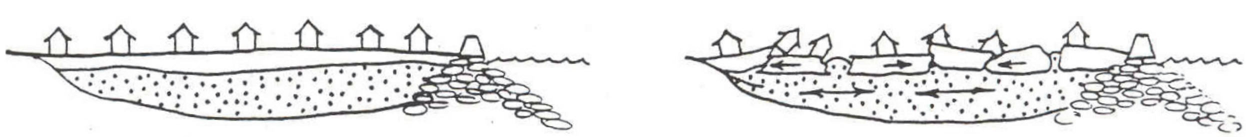
Flow failure occurs when the post-liquefaction shear strength of the liquefied soil is less than the shear stress required for static equilibrium of the system. If the soil shear strength drops below the static shear stress, flow failure occurs in which the ground will deform until it repositions itself into a configuration with lower shear stresses matching the soil strength. Resulting shear deformations are typically large (i.e., large translational or rotational failures) and often occur shortly after the conclusion of earthquake shaking. Examples of this type of failure are presented by Seed (1987). Cyclic mobility occurs when the post-liquefaction shear strength is greater than the static shear stress. The ground may "lurch" during strong pulses of motion when the shear strength is temporarily exceeded, resulting in accumulated deformations. These deformations develop incrementally during earthquake shaking in the direction of the driving static shear stress, or in the absence of static shear stresses, large transient ground oscillations may occur. Cyclic mobility can cause significant deformations of foundations, retaining walls, and slopes. Cyclic mobility of slopes or level ground behind a free face is often referred to as lateral spreading. In the case of ground oscillations without a driving shear stress, level-ground liquefaction may be observed, where ground deformations are caused by the dissipation of excess pore pressure, resulting in potential settlement and development of sand boils.



(a) Spreading towards a free face



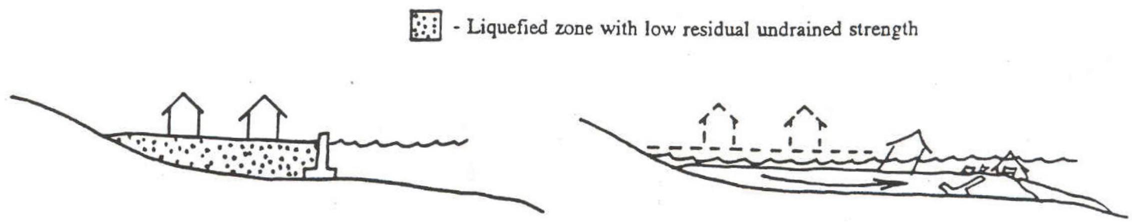
(b) Spreading downslope or downgrade



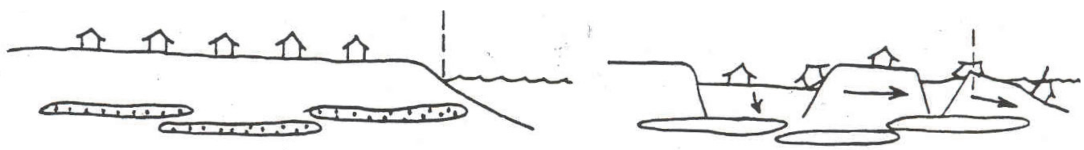
(c) Localized, non-directionally preferential differential later displacements

- Liquefied Zone

Figure 5-4 Schematic illustration of liquefaction-induced lateral displacements (Seed et al., 2001).



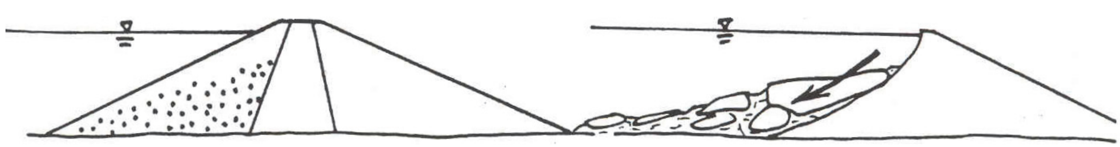
(a) Edge failure/lateral spreading by flow



(b) Edge failure/lateral spreading by translation

(c) Flow failure

(d) Translational displacement



(e) Rotational and/or translational sliding

Figure 5-5 Schematic illustrations of liquefaction-induced instabilities (Seed et al., 2001).

Unlike some other forms of earthquake-induced permanent ground deformation, liquefaction may or may not result in significant permanent deformations of the ground surface. Ishihara (1985) and Youd and Garris (1995) found through detailed analysis of field case history data that the occurrence of liquefaction in some layers of a deposit is not necessarily associated with damage to structures and disruption of the ground surface. Ishihara states, “Only when the development of liquefaction is sufficiently extensive through the depth of a deposit and shallow enough in proximity to the ground surface, do the effects of liquefaction become disastrous, leading to sand boiling and ground fissuring with various types of associated damage to structures and underground installations” (Ishihara, 1985).

Ishihara (1985) investigated the conditions under which liquefaction effects are manifest at the ground surface in terms of the thickness of liquefiable strata and overlying non-liquefiable strata. A widely used outcome of these analyses is the boundary curves shown in Figure 5-6. Using a larger data set than that of Ishihara (1985), Youd and Garris (1995) found the boundary curves in Figure 5-6 are appropriate for sites not subject to ground oscillation or lateral spread. Criteria for evaluating the potential for lateral spreading are provided in Chapter 7. Sites are likely to be subject to ground oscillation if they have laterally continuous liquefiable strata that enable decoupling of the surface soil layers from the liquefiable strata. In addition, Figure 5-6 applies essentially for sandy soils. Its reliability for fine-grained materials has not been verified, and such verification work remains a research need.

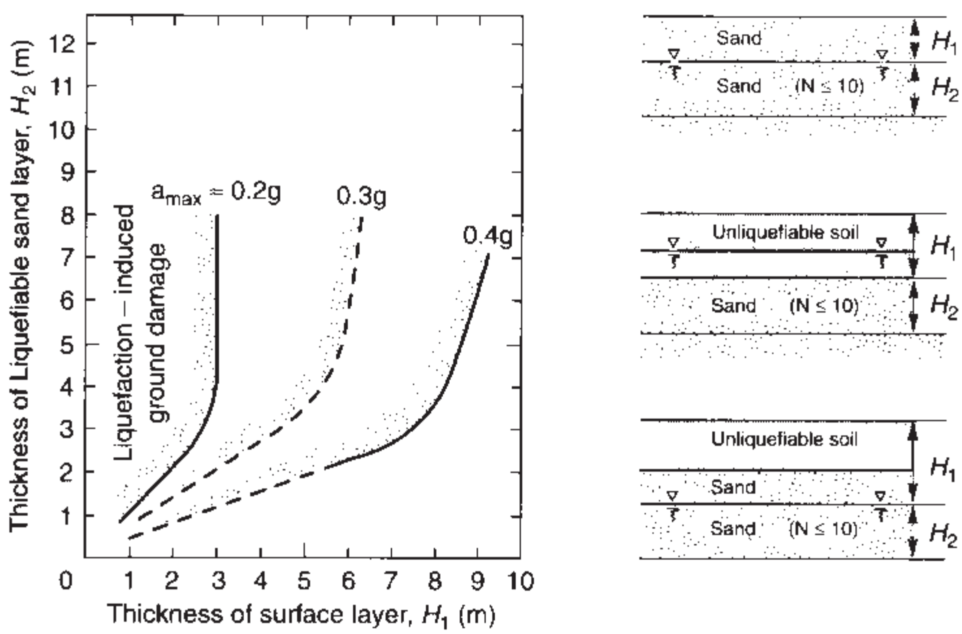


Figure 5-6 Boundary curves for surface manifestation of liquefaction at different levels of peak ground acceleration, a_{max} (Ishihara, 1985; Kramer, 1996).

When liquefaction effects are manifest at the ground surface, such effects may take the form of sand boils and ejecta, settlement, ground fissures, lateral spreads, and landslides (e.g., flow failures). In the absence of driving static shear stresses, liquefaction can result in large transient ground oscillations that can lead to buckled or cracked pavements and curbs and broken pipelines (Figure 5-7). Such damage will typically not have a consistent pattern of lateral displacements.

When driving static shear stresses are present, liquefaction can lead to lateral spreading from cyclic mobility or a flow slide (Figure 5-8 and Figure 5-9). Surface effects generated by lateral spreads will involve a consistent pattern of lateral ground movement and surface cracking (Youd and Garris, 1995). Post-liquefaction re-consolidation will occur regardless of the static shear stress condition and can result in relatively uniform settlement (Figure 5-10) or can produce differential settlements (Figure 5-11). Liquefaction at shallow depths can cause sand boils to form (Figure 5-12), which occurs when excess pore pressures in a liquefiable strata vent through cracks or openings in overlying non-liquefied layers.

Figure 5-7 Buckled pavement from liquefaction-induced ground oscillations, San Francisco Marina District, 1989 Loma Prieta, California earthquake (Seed et al., 1990).

Figure 5-8 Ground cracking of fill from liquefaction-induced cyclic mobility of foundation soils during 2001 Gujarat, India earthquake (photo credit: J. P. Stewart).

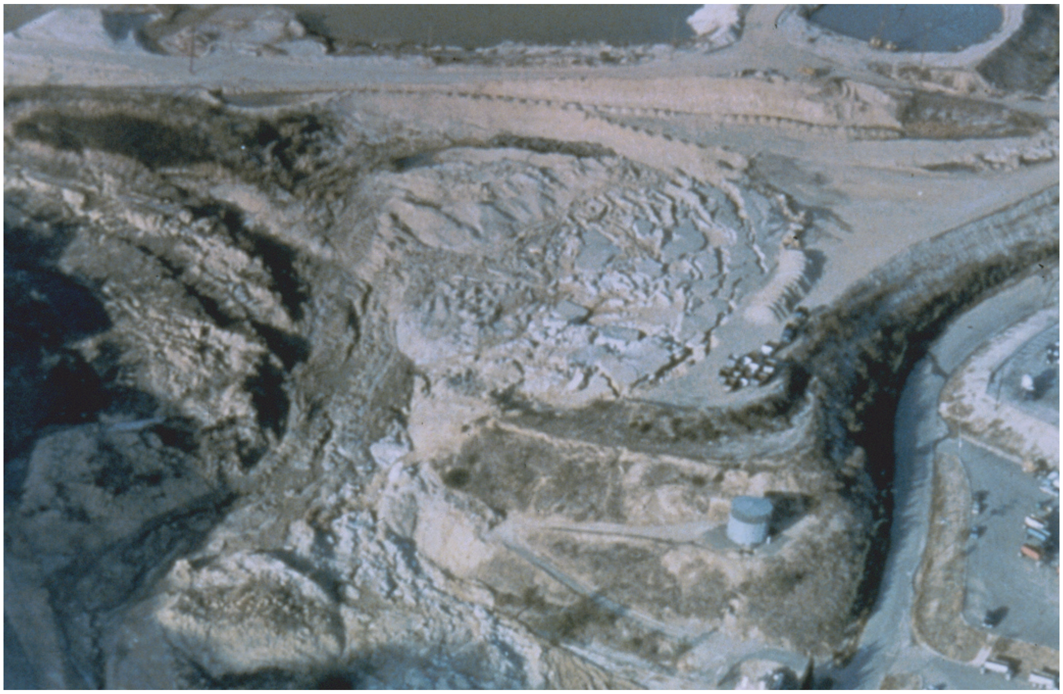


Figure 5-9 Flow slide at Tapo Canyon Tailings Dam, California, 1994 Northridge earthquake (photo credit: Y. Moriwaki).

Figure 5-10 Uniform settlement of liquefied fill soils, 1995 Kobe, Japan earthquake. The amount of settlement is shown by the difference between the unsettled pile-supported bridge bent and the surrounding ground. The uniformity of the settlement is shown by the lack of cracking in the pavement (Sitar et al., 1995).

Figure 5-11 Templeton Road Bridge settlement following the 2003 San Simeon, California earthquake. The settlement of approximately 5 inches was a result of liquefaction beneath the bridge abutment foundation (EERI, 2004).

Figure 5-12 Sand ejecta after liquefaction of hydraulic fill from the 1989 Loma Prieta, California earthquake (photo credit: NISEE).

For structures supported on shallow foundations, liquefaction effects can include foundation bearing failures (Figure 5-13), foundation settlement or tilting or both (Figure 5-14), and lateral translations of foundations (Figure 5-15). Depending on their stiffness and strength and the magnitude of deformations, the foundation elements themselves may or may not be structurally damaged as these deformations occur.

Simplified methods for liquefaction assessment were initially developed for free-field conditions and did not consider explicitly the interaction between the soil and foundation system. Research has incorporated the effects of multiple mechanisms on building settlement: volumetric settlement, shear-induced settlement, and ground loss due to ejecta (Dashti et al., 2010a; Bray and Macedo 2017). Bray and Macedo (2017) presents a simplified method for estimating building-adjacent settlement due to liquefaction that has been calibrated based on case histories, including the 2010-2011 Canterbury, New Zealand earthquake sequence.

Advances in liquefaction assessment have also moved towards recognizing the probabilistic nature of this phenomenon and an improved quantification of uncertainty (Kramer, 2018; Kramer and Mayfield, 2007), both in estimation of liquefaction triggering (i.e., soil's resistance to liquefaction and the seismic demand imposed by the earthquake) and liquefaction consequences (e.g., settlement and lateral spreading).

Figure 5-13 Foundation bearing failure of shallow foundation in Adapazari from the 1999 Kocaeli, Turkey earthquake (photo credit: J. P. Stewart).

Figure 5-14 Settlement relative to surrounding ground (foreground) of mat foundation in Adapazari from the 1999 Kocaeli, Turkey earthquake (photo credit: J. P. Stewart).

Figure 5-15 Damage to a residence in Oceano, California from lateral spreading following the 2003 San Simeon earthquake. Photo (right) taken along head scarp offset and its intersection with residence foundation (photo credits: San Luis Obispo County Department of Planning & Building, left; Holzer et al., 2004, right).

5.4 Seismic Compression

Unsaturated soil subject to large transient shear stresses can experience volumetric strains. This process is referred to by several terms, including "seismically-induced volumetric settlement," "cyclic densification," or "seismic compression." This document will use the term "seismic compression."

Seismic compression is defined as the accrual of contractive volumetric strains in unsaturated soil during strong shaking from earthquakes. Such strains may result in ground surface settlements and potential lateral movements near slopes (Figure 5-16). This process has been observed to be especially prevalent in artificial fill soils. Ground deformations in compacted fill slopes from seismic compression are well documented in the literature (Pyke et al., 1975; Stewart et al., 2001; Stewart et al., 2004a) and are recognized as representing a significant hazard with respect to collateral loss during earthquakes (SSC, 1995).

Figure 5-16 Structure damaged by movements in wedge fill from the 1994 Northridge, California earthquake (photo credit: NISEE).

Stewart et al. (2001) documented numerous case histories of ground deformations in fill from the 1994 Northridge, California earthquake. Characteristic fill deformation features are illustrated in Figure 5-17 and include cracks at cut/fill contacts due to differential settlement, ground cracks due to differential settlement across the surface of fill pads, and ground cracks due to lateral extension of fill pads towards the slope face.

The requisite conditions for seismic compression are the presence of unsaturated soil and large amplitude earthquake ground motions. Both natural and compacted fill soils can be susceptible to ground deformations from seismic compression. However, relatively few observations of seismic compression in natural soils are documented in the literature, and some existing analysis procedures apply strictly only to the seismic compression of fill materials.

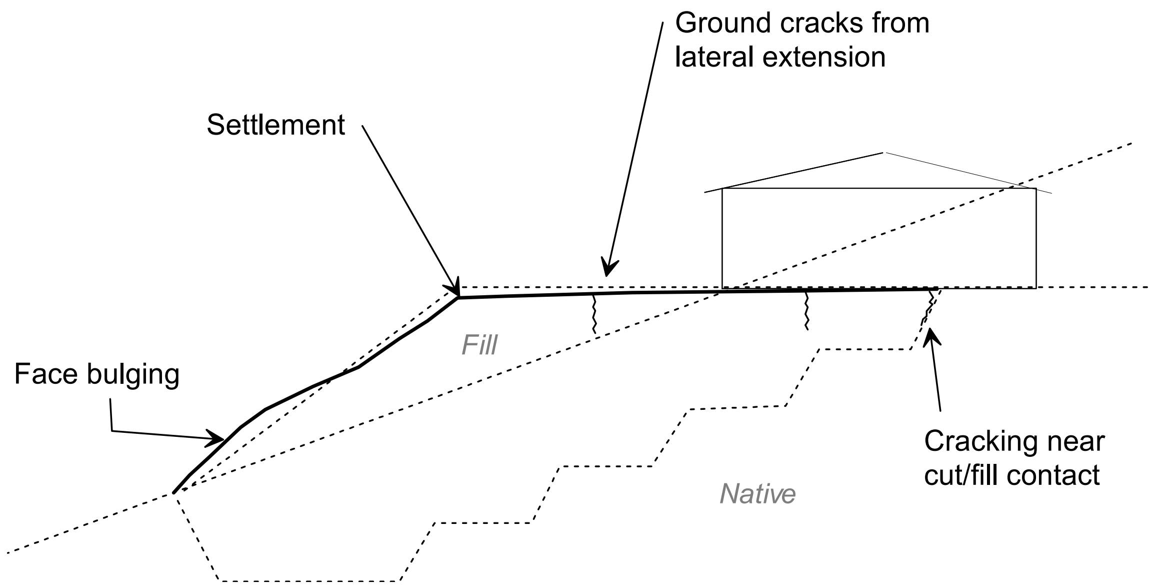


Figure 5-17 Schematic showing typical damage to fill slope (Stewart et al., 2001).

5.5 Landsliding

Inertial forces generated by strong shaking of earth slopes can cause transient shear stresses, which are time varying and only present during shaking, to develop along potential slip surfaces. When added to long-term static shear stresses, these transient stresses may cause the strength of the slope materials to be temporarily exceeded. This process leads to permanent shear deformations, which remain after earthquake shaking has stopped, within the slope materials and is referred to as earthquake-induced landsliding. Shear deformations at the base of the slide mass may be localized along a basal slip surface or may be relatively distributed across broadly stressed zones. Liquefaction-induced lateral spreading is one example of landsliding, although lateral-spread displacements are generally evaluated through a different set of procedures than those used for conventional landsliding.

Significant permanent ground deformations during earthquakes may result from landsliding. These shear deformations should be distinguished from ground settlements associated with volumetric strains that arise from post-liquefaction pore pressure dissipation or seismic compression. Earth slopes strongly shaken during earthquakes can be subject to surface displacements from both shear and volumetric strain accumulation. The subject of this section (landslides) is related to the shear deformation problem; volumetric strains are covered separately in the sections related to liquefaction and seismic compression.

In this document, the term "landsliding" is used to denote the movement of a mass of rock, earth, or debris down a slope (Cruden, 1991). Whether induced by earthquakes or other processes, landslides can be subdivided into several generalized categories (Varnes, 1978; Keefer, 1984):

- masses of disrupted slide material, such as rock falls or avalanches (Figure 5-18)
- relatively coherent slide masses whose displacement is accommodated along well-defined slip surfaces or across relatively broad, distributed shear zones
- lateral spreads and flows associated with soil strength loss due to pore pressure increase

Figure 5-18 Rockfall from the 1970 Peru earthquake (photo credit: NISEE).

Local geologic, hydrologic, and topographic conditions provide the principal means of evaluating which type of landslide mechanism is most likely for a given site. This step is crucial in engineering analyses of slope stability because different analysis procedures are appropriate for different landslide mechanisms.

Disrupted slides and falls, as described by Keefer (1984), occur in areas of high topographic relief (slopes steeper than 35 degrees to 40 degrees) and tend to involve closely jointed or weakly cemented materials (Figure 5-19). Rock avalanches are a particularly damaging type of disrupted slide, involving slide masses that originate in steep terrain and disintegrate into streams of rock that can travel large distances (on the order of miles) at high velocities. A critically important feature of many disrupted rock/soil falls is a significant loss of shear strength upon initiation of slide movement. This loss of shear strength is a characteristic feature of cemented materials and has important implications for analysis (as discussed in Chapter 7).

Figure 5-19 Example of disrupted landslide—rockfall in central Taiwan from 1999 Chi Chi, Taiwan earthquake (photo credit: J. P. Bardet).

Coherent slides can occur in rock or soil materials and at slope angles much lower than those for disrupted slides and falls (Figure 5-20). Coherent slides in rock typically involve slip along basal surfaces weakened by weathering, jointing, or prior shearing, or along bedding planes and other discontinuities that dip out of slope. Keefer (1984) reports that coherent slides in rock masses have occurred on slopes as shallow as 15 degrees. Coherent slides in soil can occur along basal slip surfaces or relatively distributed shear zones. These slides most commonly involve fill embankments (i.e., sliding occurring within the embankment materials or in relatively soft foundation soils) (Rogers, 1992; Bardet et al., 2002) but have also been widely documented in natural alluvial soils (Keefer, 1984).

Lateral spreads and flows can occur in soil on very mild slopes or behind a free-face if the soil is geologically young, has a granular texture, and the groundwater table occurs at shallow depths. The principal technical issues associated with these types of slides are related to the triggering of liquefaction and the estimation of post-liquefaction residual strengths. Both of these issues are addressed in Chapter 7. If these post-liquefaction strengths exceed static shear stresses, the problem is one of cyclic mobility, which in a slope stability context is analogous to lateral spreads. If the post-liquefaction strengths are less than static shears stresses, flow slides will occur that can involve very large displacements, such as shown in Figure 5-9.

Figure 5-20 Head scarp of coherent landslide at Junliau Switching Station, Taiwan, induced by 1999 Chi Chi, Taiwan earthquake. Slide movements of approximately 3 feet to 4 feet occurred along a bedding plane in rock (photo credit: J. P. Stewart).

Ridgetop fissuring and shattering is a result of amplification or focusing of seismic energy due to local topographic effects. This phenomenon is common on narrow ridges within steeply dipping sedimentary rock and does not necessarily indicate a slope failure has occurred (Barrows et al., 1995). Where thin soil caps overlie a ridge where fissuring or shattering has occurred, the ground surface may resemble plowed ground or be disrupted into chunks or blocks of soil. The features typically possess extensional displacements, commonly with vertical and lateral components, and may contain characteristics similar to those caused by primary or secondary surface fault rupture, lateral spreading, or landsliding (Hart et al., 1990).

5.6 Retaining Wall Deformation

A retaining wall is defined as any wall that retains soil or rock to maintain a change in elevation (ASCE, 1994a) (Figure 5-21). The function of retaining walls is to safely support the retained material and any structures constructed behind the wall (e.g., soil slope, building, roadway) without excessive deformation. In service, most retaining walls deform to some degree. When retaining wall deformation, whether earthquake induced or otherwise, becomes excessive, the retaining wall is said to have “failed.” However, with the exception of obvious or imminent collapse, the magnitude of retaining wall deformation that constitutes failure, or even damage, has not been well defined.

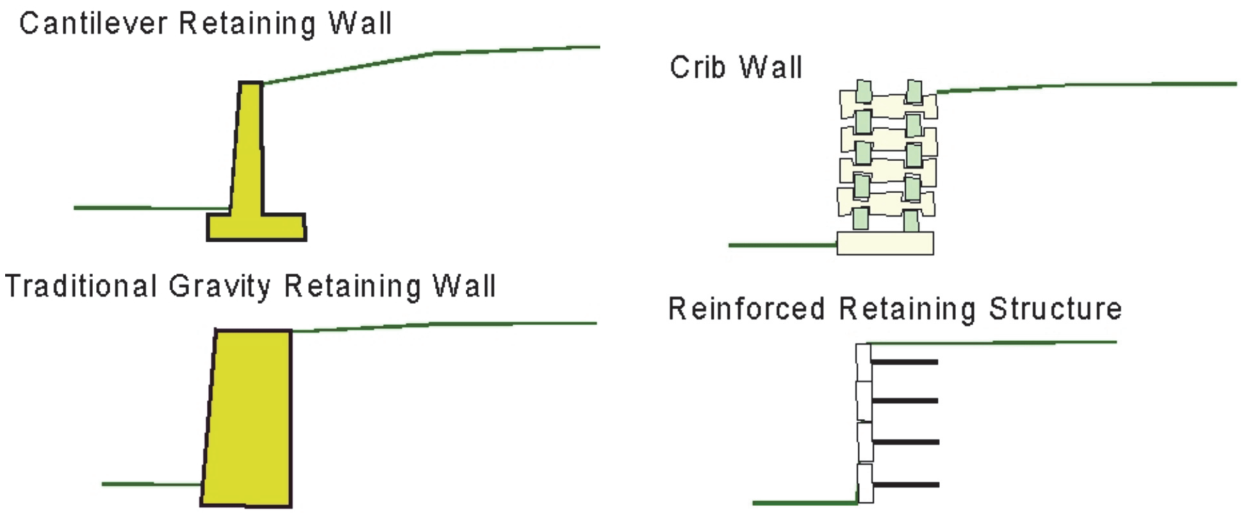


Figure 5-21 Examples of different types of retaining walls utilized in residential construction (image credit: Exponent).

Earthquake-induced retaining wall deformations may result from either permanent ground deformation or loading of the wall. Earthquake-induced permanent ground deformation may cause deformations (i.e., settlement, lateral movement, global instability) in the supporting ground that produce retaining wall movement independently of the earthquake loading on the wall. These mechanisms of permanent ground deformation are discussed in the preceding sections. The focus of this section and related sections in Chapter 6 and Chapter 7 is deformations of residential retaining structures resulting solely from earthquake-induced loading of the wall that may have altered the stability or serviceability of the wall.

During earthquake shaking, cantilever and gravity retaining walls may move by sliding or tilting (Figure 5-22). Reinforced soil slopes and walls deform in a ductile manner without formation of a distinct failure surface and may produce minor settlement of the backfill, face bulging or spalling, and minor cracking in the backfill. The magnitude of movement is related to soil conditions (e.g., backfill properties), design of the wall, and, in the case of reinforced soil slopes or walls, slope inclination and reinforcement stiffness and spacing (Nova-Roessig, 1999; Siddharthan et al., 1992). Dynamic wall pressures are influenced by the magnitude of the earthquake ground accelerations, and the dynamic response and natural frequency of the wall-backfill system (Nadim, 1982; Whitman, 1990).

Post-earthquake evaluation of retaining walls requires evaluation of the stability, serviceability, and appearance with respect to the nature and extent of wall deformations. Post-earthquake serviceability of retaining walls is closely related to the total permanent deformations that the wall has experienced from seismic movements and non-earthquake induced movements. The focus of this section and related sections in Chapter 6 and Chapter 7 is on distinguishing between conditions that may have been earthquake induced and those conditions resulting from long-term processes. Chapter 6 and Chapter 7 also describe assessing whether the earthquake-induced conditions have damaged or caused permanent deformation of the wall.

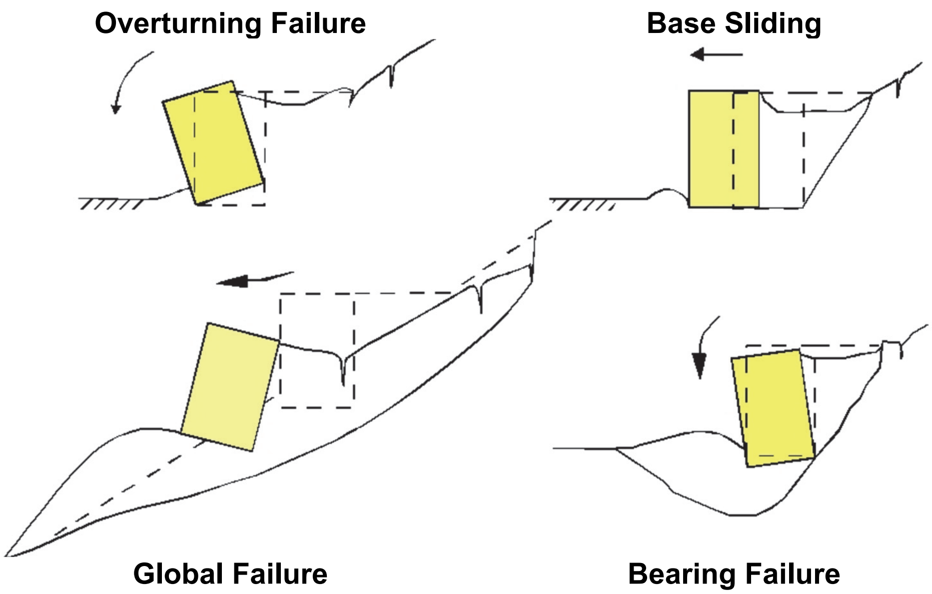


Figure 5-22 Examples of modes of failure of retaining walls (image credit: Exponent).

Retaining wall damage is defined as conditions that reduce the wall's stability below minimum requirements under reasonable future loading conditions, that materially alter its serviceability, or that materially affect its appearance. Retaining wall analyses typically recognize that, in some instances, large permanent wall deformations may be acceptable, while in others, smaller deformations may not, and the wall may be considered damaged or even "failed" at these smaller deformations. Reasonable assumptions regarding future loading and performance expectations for the wall are essential for these analyses.

5.7 Overview of Non-Seismic Ground Deformation Processes

A variety of non-seismic, long-term geotechnical processes cause ground displacements that damage structures. Damage caused by non-seismic ground movements can appear similar to earthquake-induced ground movements. However, there is a need for engineers to distinguish between seismic and non-seismic causes of ground movement when investigating a property for earthquake damage. All buildings experience some amount of non-seismic differential movement during their service lives. Often this movement is caused by soil deformations beneath and around the foundation, while some differential movements occur in the structure due to normal shrinkage or swelling of structural members, and thermal expansion or contraction of finishes. These movements are commonly manifested in the superstructure as finish cracking, foundation cracking, and uneven floors.

During post-earthquake investigation of a flat site, a technical consultant will need to distinguish ground settlements or heave or both that are characteristic of static volume change phenomena from ground settlements associated with liquefaction or seismic compression. Similarly, investigations of sloping sites should distinguish long-term slope instability (i.e., landslides), creep, or retaining wall movements from ground deformations associated with earthquake-induced landslides or retaining walls movement. The most common non-seismic mechanisms of ground deformation or failure were listed at the beginning of this chapter as consolidation settlement, immediate settlement, hydro-compression settlement, expansive soil movement, slope creep, landsliding, and retaining wall deformation. For each of these mechanisms of deformation, the following sections provide description of the phenomena, visual indications that distinguish the deformation from earthquake-induced deformation, and briefly discuss investigative methods.

5.7.1 Consolidation Settlement

Consolidation settlement is defined as volume change due to dissipation of excess pore pressure, defined as pore pressures beyond the hydrostatic pore pressure, resulting in expulsion of water from the soil matrix and increased effective stress. Discussions of consolidation settlements may be found in Terzaghi et al. (1996), Holtz and Kovacs (1981), Mitchell (1993), and ASCE (1994b). The rate of consolidation settlement is dependent upon soil properties and the length of the drainage path, and the magnitude of settlement depends on site conditions. Consolidation occurs quickly in coarse-grained soils, such as sands and gravels, because these soils have relatively large permeabilities, hence these settlements are usually not distinguishable from immediate settlements discussed below. Consolidation in fine-grained soils, such as clays, silts, and organic materials, can be significant and take considerably longer to complete. The excess pore pressures responsible for consolidation can result from changes in overburden pressure (for example, following fill placement or the addition of structural loads) or changes in ground water levels.

Consolidation settlements occur most rapidly after construction. As a result, damage associated with these settlements occurs during the period when consolidation is occurring. The indicators that consolidation settlements have occurred or are occurring at a site are uneven floors, finish distress (typically repaired in older homes) in areas of floor elevation gradients, and doors and windows that stick in their frames or be difficult to operate. The pattern of uneven floors typically will indicate the largest settlement beneath areas of greatest foundation load or areas with thickest deposits of soil experiencing consolidation settlements.

Investigations of consolidation settlement typically include laboratory testing of minimally disturbed samples retrieved from test pits or from drilling of boreholes. Soil sample testing may include consolidation testing according to ASTM D 2435. Results of the laboratory testing can be utilized to estimate the magnitude and rate of consolidation testing for the soil and loads encountered in the field (Terzaghi et al., 1996; Fang, 1991; ASCE, 1994b).

5.7.2 Immediate Settlement

Immediate settlement is defined as settlement caused by small-strain shear or volumetric deformations or both in soil that are not associated with consolidation or hydro-compression (discussed in the following section). These deformations are sometimes referred to as elastic settlements. A common example of this phenomenon is young fills that compress under their own weight and surface loading prior to the introduction of water. If the level of saturation in the fill is low, the volume reduction is not associated with pore pressure dissipation and instead depends principally on the bulk and shear moduli of the soil.

Immediate settlements occur essentially at the same time loads are applied to the soil. Damage from these types of settlements therefore occur very early in the life of the structure. Typical damage includes uneven floors and cracks in finishes and foundations.

Immediate settlements in cohesionless soil is complicated by a nonlinear stiffness that depends on the state of stress. Empirical and semi-empirical methods for calculating immediate settlements can be found in ASCE (1994b). Immediate settlements in cohesive soil can be estimated using elastic theory and are discussed in most geotechnical engineering texts (Terzaghi et al., 1996; Fang, 1991; Lambe and Whitman, 1969; Bowles, 1996).

5.7.3 Hydro-Compression Settlement

Hydro-compression settlement is volume reduction of unsaturated soils upon wetting, which is associated with collapse of the soil fabric. Soils subject to collapse can include wind-deposited (aeolian) sands and silts, alluvial fan and mudflow sediments, and some man-made fills. The common characteristics of these soils is a loose structure and large void ratio. Volume reductions are rapid upon introduction of water; however, settlements will occur over time until all the collapse potential is achieved through wetting. The rate of settlement depends on the rate of water infiltration into the soil.

The cohesion in collapsible soils is usually provided by capillary tension of pore water or chemical bonding of particles with soluble compounds such as salts. Collapse occurs as water is added due to the reduction of capillary tension or the weakening of the chemical bonds (particularly acidic water) or both. These soils are often strong and stable when dry. The magnitude of settlement resulting from collapsible soils depends on the initial void ratio, stress history of the soil, thickness of the collapsible layer, and magnitude of the overburden pressure. Areas with collapsible soils exposed to irrigation along the perimeter of a residence, focused runoff such as from downspouts, or leaking utility lines are most likely to settle. Other causes of collapsible soil settlement include a slow uniform rise in groundwater.

Manifestations of collapsible soils are non-uniform settlements of the foundation, adjacent ground surface, and surface improvements, such as sidewalks and patios. These exterior areas will often be depressed sufficiently to pond water during rain events. Damage to residences typically includes localized uneven floors, cracks in wall finishes and foundations, and difficulty operating doors and windows. The damage and uneven floors are highly correlated. The pattern of ground settlement across a site affected by hydro-compression will typically be closely related to the variation in thickness of the susceptible soils (e.g., compacted fills).

Typical collapsible soils are low in plasticity with liquid limits below 45, plasticity indices below 25, and relatively low dry densities between 65 lbs/ft^3 and 105 lbs/ft^3 (ASCE, 1994b). Investigations of hydro-compression settlement typically include laboratory testing of minimally disturbed soil samples or remolded, recompacted soil samples. In situ densities can be evaluated with downhole in situ sand cone tests (ASTM D 2419) or through laboratory testing of samples retrieved in the field (ASTM D 2937). Samples to be used for such purposes should be disturbed to the least extent practical, as disturbance will change sample density. In addition to density, soil index tests that are useful in seismic compression analyses include water content (ASTM D 2937), gradation (ASTM D 422 or ASTM D 1140), and liquid limit and plastic limit (ASTM D 4318). Testing of collapsible potential can be performed by several methods, such as oedometer tests (ASTM D 4546) or via a modified oedometer test as described in ASCE (1994b).

5.7.4 *Expansive Soil Movement*

Expansive soil movement is defined as shrink or swell of plastic clays when the water content is reduced (i.e., drying) or increased (i.e., wetting). Clays, particularly those containing montmorillonite or smectite minerals, are sensitive to water content changes. Cycles of shrinking and swelling typically occur in near-surface soil layers subjected to transient water content fluctuations. The water content variation can be seasonal (e.g., summer to winter) or can follow a long-term trend (e.g., from changes in landscaping and vegetation or installation of pavements that change surface drainage patterns) or may be more transient, such as from irrigation or utility line leaks. The soil depth above which changes in water content and soil heave or shrinkage can occur because of changes in environmental conditions is termed the active zone.

A good indication that expansive soils are present at a site is desiccation cracks in the soil surface. Typical damage to residential structures from expansive soil movement are uneven floors, cracks in wall finishes, cracks in foundations and floor slabs, difficulty operating doors and windows, and cracking of flatwork with offsets across the cracks. The location of damage within a residence is dependent upon several factors, such as drainage conditions around the residence, location of vegetation, and irrigation patterns. A distinguishing characteristic of damage from expansive soil movement is repeated patching and painting of distressed areas as a result of the cyclic nature of the movement. See Section 3.5.2 of the *General Guidelines* for figures depicting typical damage from expansive soil movement on residential foundation systems.

Discussions of expansive soil behavior and the effect of vegetation may be found in ASCE (1994b), Institute of Civil Engineers (1984), and BRE (1985). Identification of expansive soils can be performed with standard laboratory testing, such as liquid limit and plastic limit (ASTM D 4318), expansion index (ASTM D 4829 and UBC Standard No. 29-2), or oedometer tests (ASTM D 4546). Guidelines for classifying expansive soil may be found in Snethen et al. (1977). Quantification of volume change and swell pressures may be found in ASCE (1994b).

5.7.5 *Slope Creep*

Slope creep is defined as slow downslope movement of plastic rock and soil materials. Some creep takes place in almost all steep earth and rock slopes. Slope creep is associated with deformation without volume and pore water pressure changes in a soil subject to shear; this type of deformation is also referred to as shear creep. The rate of creep is dependent on factors such as material type, slope inclination, and water content fluctuations within the slope.

Creep may be classified as seasonal creep and continuous creep (Fang, 1991). Seasonal creep is caused by temperature (i.e., freezing and thawing) and water content variations in the soil and rock within the surface layer. Seasonal creep primarily takes place in clayey and silty soils. Typically, the depth of the surface layer affected by seasonal creep is less than the depth of seasonal temperature and water content variations. Expansive soils on a slope will creep over time as a result of the swelling, which occurs perpendicular to the slope face, and shrinkage, which occurs vertically in the direction of gravity. Continuous creep is caused by gravitational forces and is differentiated from seasonal creep by the depth of soil affected, which is typically deeper than the zone of seasonal water content fluctuations. Further discussion of creep behavior and modeling may be found in Mitchell (1993) and Mitchell and Soga (2005).

Slope creep occurs within shallow soil or rock materials, and hence damage from slope creep is generally confined to areas along a slope face or near the top of slope. Slope creep is a process that fundamentally is time dependent and therefore damage manifesting from this type of movement appears over time. Indicators of creep movement are uneven floors, damaged wall finishes, extensional features in at-grade improvements near the top of slope (such as separations in flatwork from the residence), and tilting of support posts for decks and floors. Because of its time-dependent nature, damage resulting from creep is often characterized by repeated repairs of improvements. Figure 5-23 shows a schematic representation of damage that can result from creep.

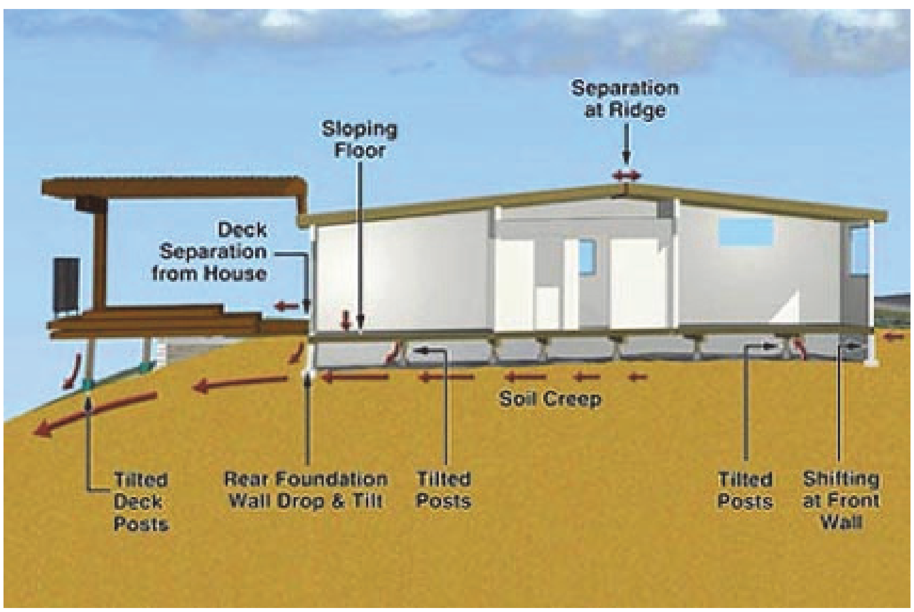


Figure 5-23 Schematic of damage to residence resulting from soil creep (figure credit: Exponent).

5.7.6 Non-Seismic Landsliding

The term "landslide" encompasses a wide range of ground movements, such as shallow rock falls, deep-seated slope failures, and flow slides, such as earth or debris flows. Other than from earthquakes, landslides can be triggered by numerous other processes including, but not limited to, changes in slope geometry (i.e., excavation near slope toe), loading of the top of slope, and increased water pressure within the slope.

5.7.7 Non-Seismic Retaining Wall Deformation and Failure

Non-seismic issues with retaining walls can include tilt, sliding, deterioration, and failure. Excessive movements of retaining walls can result in soil deformations and ground cracking behind the walls. Retaining wall failures most often occur because the walls experience lateral earth pressures beyond their capacity (e.g., from slope creep or poor drainage).

CHAPTER 6

Damage Investigation: Geotechnical

6.1 Introduction

This chapter provides guidelines for post-earthquake site investigation to evaluate the potential occurrence of earthquake-induced permanent ground deformation phenomena. The emphasis is on the issues raised in Chapter 5—distinguishing earthquake-induced permanent ground deformation phenomena from pre-earthquake ground deformations caused by long-term (non-seismic) geotechnical processes. Discussions of recommended equipment and general safety practices are outside the scope of this document.

A site-specific investigation of potential earthquake-induced permanent ground deformation typically includes two parts: *literature review* and *field reconnaissance*. The literature review entails the study of published information on the earthquake and the geologic and geotechnical conditions at the site and surrounding area. The field reconnaissance consists of observations and documentation at the site and surrounding area. These components of the site investigation are discussed in the sections below.

6.2 Literature Review

The principal objective of the literature review is to gather and distill data that supplement and provide context for the observations documented during field reconnaissance. For post-earthquake investigation of potential earthquake-induced permanent ground deformation, the scope of the literature review evolves over time as more information pertaining to an earthquake is published. For example, following an earthquake, literature becomes available regarding field reconnaissance observations, ground motion data, structure damage reports, and damage distribution. Sources of this information can include government agencies, universities, and professional associations, such as geotechnical studies by the Geotechnical Extreme Events Reconnaissance (GEER) Association. These publications typically include maps delineating observations of permanent ground deformation. However, unless the specific site under consideration was visited by a reconnaissance team member, the results of site-specific reconnaissance should be considered more reliable than the findings in a regional reconnaissance report.

Additional sources of information typically consulted prior to performing field work include seismic hazard maps for faulting, liquefaction, and landslide hazards, topographic maps, geologic and soils engineering maps and reports, agricultural soil survey maps, ground water contour maps, and aerial photographs.

The U.S. Geological Survey (USGS) Earthquake Hazards Program website typically provides current information regarding recent earthquakes and often compiles information from local agencies in the summary for a given earthquake event (e.g., for the 2018 Anchorage, Alaska, earthquake, the USGS event summary also provided information from the Alaska Earthquake Center).

6.3 Field Reconnaissance

The principal objective of geotechnical field reconnaissance is to identify features at a site that may indicate earthquake-induced permanent ground deformation or static ground deformation mechanisms. Field reconnaissance occurs primarily on two scales: *regional* and *site specific*. In the following sections, the objectives of each are described, along with the ground deformation features that geotechnical consultants may observe.

Field reconnaissance is best performed shortly after the earthquake, while ground deformation features are still evident. Field reconnaissance typically consists of ground mapping and aerial overflights, but in recent years the use of drones has allowed for relatively rapid field reconnaissance of large areas. The use of drones can be beneficial for scanning large areas and identifying features for ground mapping and further investigation. Drone surveys can also be useful in areas that are not readily accessible or that may be subject to instability (e.g., hillside properties with potential slope instabilities). Use of drones should comply with all local, state, and federal rules governing their operation. As with any site investigation, appropriate field reconnaissance techniques will depend on the site configuration and sound engineering judgment as to the most appropriate techniques.

Media sources (e.g., local news and social media) can provide anecdotal information that may aid a field reconnaissance effort. Scientific or engineering evidence and explanation from media sources should be independently verified.

Evaluation of potential earthquake-induced permanent ground deformation at a site begins with a thorough site reconnaissance. An essential part of reconnaissance is an examination of the general vicinity of the site, the site itself, and improvements on the site for indications of potential earthquake-induced permanent ground deformation. Reconnaissance should be performed as soon as practical after the earthquake, so that cracks in the soil or flatwork will be "fresh" and thus readily distinguishable from cracks that may have pre-existed the earthquake. The level of effort during reconnaissance depends on how readily observable and unambiguous the indicators of earthquake-induced permanent ground deformation are at a site.

Following field reconnaissance, additional investigations may be necessary. Geotechnical investigations involving subsurface exploration, laboratory testing, and engineering analyses are typically only needed under the following circumstances:

- Indicators of earthquake-induced permanent ground deformation are present at the site. In this case, geotechnical investigations and analyses may be needed to determine the nature and extent of the earthquake-induced permanent ground deformation and to design appropriate repairs.
- Indicators of earthquake-induced permanent ground deformation are present in the immediate vicinity of the site. In this case, the investigations and analyses are needed to determine if permanent ground deformation occurred at the site.

- Potential earthquake- and non-earthquake-induced deformations or damage cannot be readily differentiated on the basis of available field data. In this case, site investigations and analyses are necessary to investigate the potential for earthquake-induced permanent ground deformation.

The central elements of field reconnaissance are discussed in the remainder of this chapter.

6.3.1 Identifying Earthquake-Induced Permanent Ground Deformation

Geotechnical consultants using this document should be familiar with and able to recognize the mechanisms of earthquake-induced permanent ground deformation. In order to facilitate recognition of earthquake-induced permanent ground deformation, examples of the mechanisms were presented in Table 5-1. The table contains a brief description of each mechanism (e.g., surface fault rupture, liquefaction) and references to example figures and photographs. Table 6-1 presents typical ground deformations that may be observed at a site and lists potential earthquake-induced causative mechanisms for those deformations. Figure 6-1 presents a schematic illustration of ground deformation that may be observed on a hillside property. Table 6-2 summarizes various regional and site-specific indicators of potential permanent ground deformations.

Table 6-1 Ground Deformation Observations and Potential Earthquake-Induced Causative Mechanisms

Ground Deformation Observation	*Potential Earthquake-Induced Causative Mechanism*
Cracking, level ground	Surface fault rupture Liquefaction Seismic compression
Extensional cracks	Slope failure (landslide, lateral spreading) Seismic compression
Sand boils and ejecta	Liquefaction
Settlement	Liquefaction Seismic compression
Vertical or horizontal offsets, block movement	Surface fault rupture Landsliding Lateral spreading
Slumping, sloughing	Landsliding Lateral spreading
Compression mound, bulging	Landsliding Liquefaction (of underlying stratum)

Note: For each of these ground deformation observations (with the exception of sand boils and ejecta), typically multiple non-seismic causative mechanisms exist that could produce the same condition. Consequently, this table is not intended to imply that observing a listed ground deformation necessarily means that the associated earthquake-induced mechanism caused the deformation.

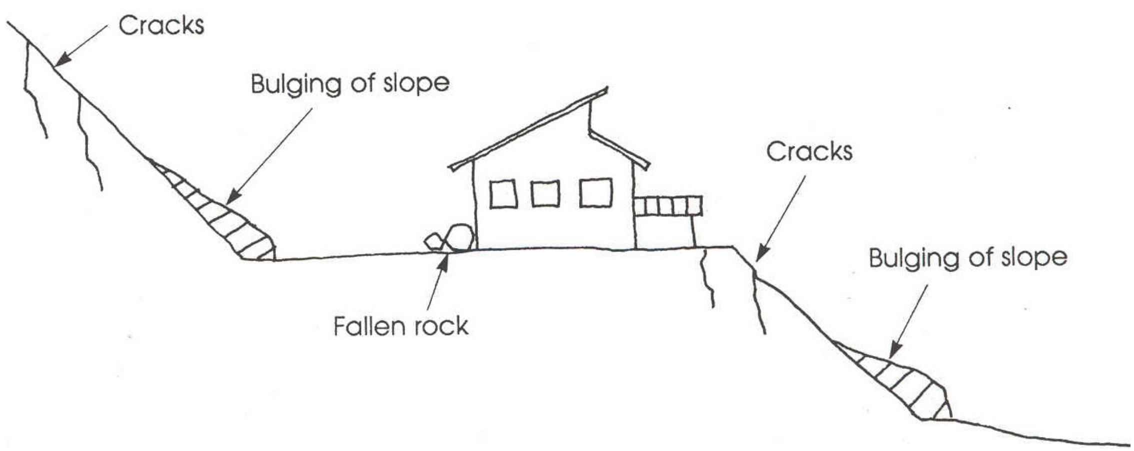


Figure 6-1 Schematic illustration of evidence of potential earthquake-induced ground failure (ATC, 2005).

Table 6-2 Indicators of Potential Permanent Ground Deformation

Condition	*Regional Indicator*	*Site-Specific Indicator*
Conspicuous evidence of permanent ground deformation	Surface manifestation of liquefaction (e.g., sand boils), landslide, flow failure, lateral spreading within a block or two of the house, linear fissures in ground parallel to a slope (not to be confused with fissuring caused by desiccation of expansive soil), rising or bulging of the ground surface, signs of slope movement (above or below or adjacent to the property), or collapse or significant rotation of retaining structures	Surface manifestation of liquefaction (e.g., sand boils), obvious evidence of slope movement such as linear fissures in ground parallel to a slope (not to be confused with fissuring caused by desiccation of expansive soil), rising or bulging of the ground surface, or collapse or significant rotation of retaining structures
Streets, curbs, and sidewalks and site flatwork condition	Unusual cracking (i.e., cracking not associated with shrinkage stresses, thermal stresses, traffic patterns, effects of vegetation like large trees, or age) or recent street patching indicating repairs of street following repairs to underground utilities by municipalities	Fresh-appearing soil cracks or flatwork cracks or separations of construction joints wider than 1/4 inch or fresh-appearing out-of-levelness in patios, decks or other flatwork inconsistent with normal construction practices (e.g., patios adjacent to structures are sloped away from the building for drainage), general level of maintenance, and normal aging. When expansive soils are present, it is common for slabs to “walk” away from the structure over time, resulting in a gap between the flatwork and foundation
Utility line condition	Occurrences of underground utility line breaks in the vicinity of the property, such as water service lines, water mains, sanitary sewer lines, gas lines (see also street condition for indicators when these lines have been repaired prior to the inspection)	Damage to underground utility lines on the property, such as water service lines, sanitary sewer lines, or gas lines
Swimming pool condition	Not applicable	Fresh-appearing out-of-levelness of the pool coping inconsistent with the general level of maintenance and normal aging, and fresh-appearing cracking of the pool shell

Table 6-2 Indicators of Potential Permanent Ground Deformation (continued)

Condition	*Regional Indicator*	*Site-Specific Indicator*
Foundation condition	Not applicable	Fresh cracks wider than 1/8 inch in concrete foundation elements, fresh-appearing foundation movement or damage, out-of-levelness consistent with conspicuous earthquake damage to adjacent house walls, fresh-appearing cracks in tile applied on a slab-on-grade surface, fresh cracks greater than 1/8 inch in stucco applied directly onto the exposed foundation edge

Note: All quantified site-specific indicators, such as cracks wider than 1/4 inch, are approximate and consistent with recommendations in the *General Guidelines*.

6.3.2 Regional Reconnaissance

The principal purpose of regional reconnaissance is to identify damage patterns and ground deformation features in the surrounding area that can provide insight into observations of permanent ground deformation (or lack thereof) at the specific site under investigation. In performing regional reconnaissance, a geotechnical consultant seeks to:

- identify patterns of damage and ground deformation that establish if permanent ground deformation occurred in the area, and
- establish a baseline earthquake intensity, such as by using the Instrumental Intensity presented in Section 2.4, for the area.

Regional reconnaissance is typically performed by driving and walking through the neighborhood of the property. During the reconnaissance, *obvious* indicators of permanent ground deformation within several blocks of the house may be present. These indicators can include:

- surface fault rupture,
- large lateral or vertical ground displacements associated with liquefaction, such as flow failure or lateral spreading,
- sand boils from liquefaction, or
- landsliding, as evidenced by rockfalls—linear fissures in ground parallel to the strike of a slope (not to be confused with fissuring caused by desiccation of expansive soil)—or rising or bulging of the ground surface near the toe of slope.

If obvious indicators of permanent ground deformation are not present, the geotechnical consultant should investigate the presence of more subtle permanent ground deformation involving relatively small ground displacements. These indicators can include:

- **Streets, curbs, and sidewalks.** Because they are relatively brittle and cover large areas, streets, curbs, and sidewalks are excellent indicators of permanent ground deformation. Unusual cracking of these improvements (i.e., cracking not associated with shrinkage stresses, thermal stresses, traffic patterns, effects of vegetation like large trees) are indications of permanent ground deformation. For example, evidence of conspicuous and unusual damage to the streets, driveways, sidewalks, curbs, or gutters near a house may indicate that permanent ground deformation occurred during the earthquake. However, an apparent random pattern of damage to these improvements, especially if the cracks are not fresh, may indicate that non-seismic deformation mechanisms, such as expansive or collapsible soils, may be present and resulting in ground movement.

- **Utility lines.** Utility lines can be sensitive to ground deformations and breaks in these lines can be a good indicator of permanent ground deformation. Examples of these utilities include water service pipes, water mains, sanitary sewer pipes, and gas pipes (Figure 6-2). If possible, modes of pipe failure should be noted (e.g., extension, shear, compression). When the reconnaissance is performed after an earthquake, repairs of underground utilities by municipalities may have already occurred; hence, recently patched areas of streets may be evidence of underground utility repair (i.e., water or sewer pipe breaks).
- **Nearby houses.** Damages to nearby properties are good indicators of the intensity of shaking experienced at a house. The condition of improvements with different vulnerabilities to earthquake damage should be noted. For example, damage to utility lines, flatwork (Figure 6-3), or foundations (Figure 6-4) at nearby houses are indicators of permanent ground deformation that warrant consideration at the subject site. However, block wall damage is common even at low intensity of earthquake shaking and is not a good indicator of earthquake-induced permanent ground deformation if the crack pattern in the wall is inconsistent with a prevailing pattern of ground deformation.

Figure 6-2 Pipe failures and resulting pavement distress and settlement from the 1971 San Fernando, California earthquake (photo credit: NISEE).

Figure 6-3 Separation in sidewalk from 1957 Daly City, California earthquake (photo credit: NISEE).

Figure 6-4 Damage to house foundation in Wakami, Japan, caused by lateral spreading following the 1983 Nihonkai-Chubu, Japan earthquake (photo credit: NISEE).

Observations from regional reconnaissance should be documented with photographs and notes to describe the condition of the surrounding area. Indicators of both earthquake-related permanent ground deformation and non-earthquake-related damage should be recorded.

6.3.3 *Site-Specific Reconnaissance*

Site-specific reconnaissance involves observations and documentation at the specific property of interest. The objectives of site-specific reconnaissance are to:

- evaluate if regional permanent ground deformation mechanisms observed in the surrounding area during regional reconnaissance are manifest at the property of interest,
- evaluate if localized permanent ground deformation with a unique character relative to the surrounding area may have occurred at the site, and
- identify additional investigation activities that may be needed to definitively identify the permanent ground deformation mechanisms, and if necessary, to facilitate the design of repair or mitigation measures.

Geotechnical consultants performing site-specific reconnaissance of a residential property should be familiar with basic residential wood-frame construction and construction of improvements typically found on residential properties, such as flatwork, retaining walls, and swimming pools. In particular, a geotechnical consultant should be familiar with the common foundation types utilized for residential structures (see Section 2.3.3). Figure 6-5 presents typical inspection points of a residential structure for evaluating if permanent ground deformation may have caused damage to a structure. In addition, minimum guidelines for inspection of geotechnical hazards can be found in ATC-20-1, *Field Manual: Postearthquake Safety Evaluation of Buildings* (ATC, 2005).

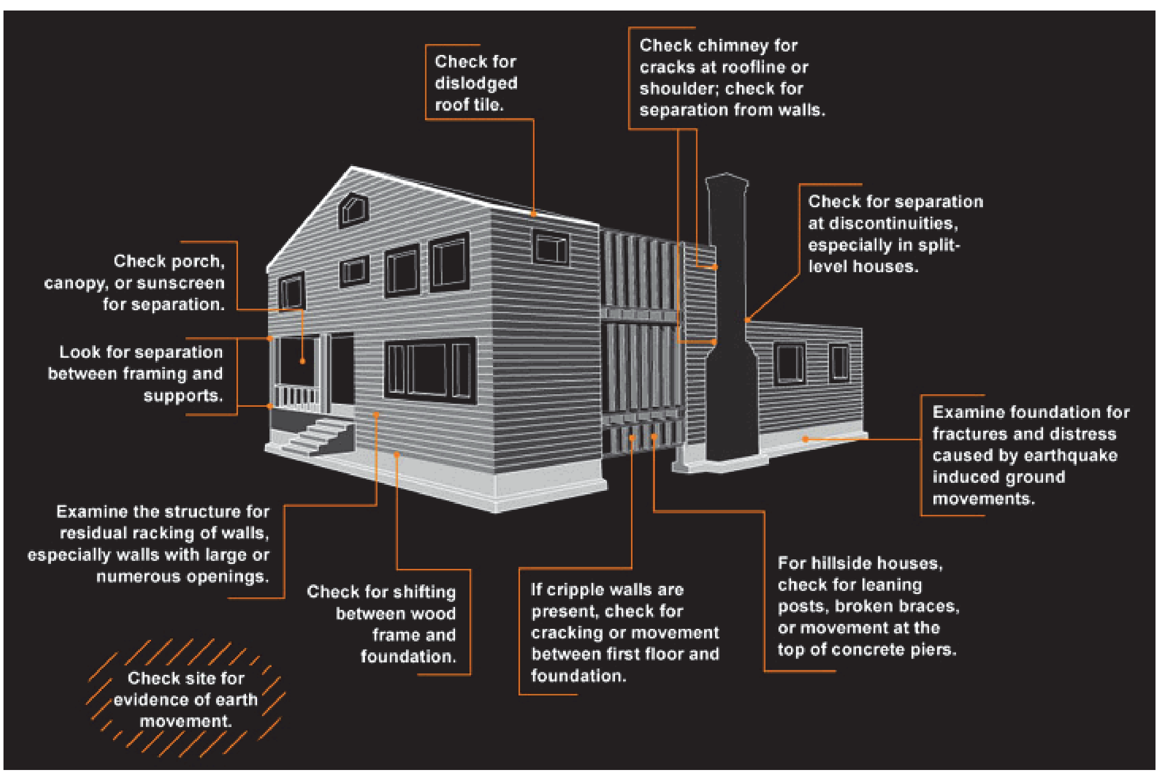


Figure 6-5 Typical inspection points during site reconnaissance (adapted from: ATC, 2005).

Site-specific field reconnaissance typically includes the following tasks:

1. Discuss the pre- and post-earthquake condition of the property with the occupant and report, based on the occupant's recollection, those damage features that pre-existed the earthquake and those that were either changed by the earthquake or newly appeared following the earthquake.
2. Perform a detailed visual inspection of the property and report significant permanent ground deformation features on a suitable map of the property with accompanying photographs of each feature. At a minimum, the condition of flatwork, block walls, foundations, pools, and structural finishes should be reported on the map. Soil surfaces should also be examined for potential deformations.
3. Report on the condition of observed cracks in soil and site improvements, like pavement. For example, crack widths should be reported along with the freshness of cracks. The presence in cracks of foreign substances, such as paint, debris, patching or releveling compounds, should be noted if present.

While performing Task 2 and Task 3, the geotechnical consultant should report *obvious* evidence of permanent ground deformation. Examples of obvious evidence include settlement, liquefaction, slope movement, rising or bulging of the ground surface, and collapse or significant rotation of retaining structures.

Whether or not these obvious indicators of permanent ground deformation are observed, the geotechnical consultant's report for the property typically documents the condition of the following components of the site:

- **Hardscape and flatwork.** Fresh cracks in hardscape or flatwork (e.g., driveways, patios, sidewalks) or the opening of construction joints are good indicators of earthquake-induced permanent ground deformation, especially if the cracking pattern is consistent with a prevailing pattern of ground deformation. Also, fresh-appearing out-of-levelness in patios, decks, or other hardscape or flatwork inconsistent with normal construction practices (e.g., patios adjacent to structures are sloped away from the building for drainage) are indicators of potential earthquake-induced permanent ground deformation. However, fresh cracks that form a random pattern may represent pre-earthquake damage that was worsened by the earthquake shaking. Indicators such as paint or debris within cracks, previous patching of cracks, and previous releveling are good indicators of pre-earthquake damage possibly associated with static ground deformation (see Figure 6-6, Figure 6-7, Figure 6-8, and Figure 6-9).
- **Utility lines.** Damage to underground utility lines on the property, such as water service lines, sanitary sewer lines, and gas lines, are indicators of permanent ground deformation. If not obviously damaged, the condition of these lines should be queried during discussions with people most knowledgeable about the post-earthquake condition of the property.

Figure 6-6 Localized compression feature and pavement damage from the 1994 Northridge, California earthquake (photo credit: NISEE).

Figure 6-7 Sidewalk buckled from lateral compression by the Northridge Hospital Medical Center from the 1994 Northridge, California earthquake (photo credit: NISEE).

Figure 6-8 Offset sidewalk from the 1971 San Fernando, California earthquake (photo credit: NISEE).

Figure 6-9 Tree root damage to sidewalks and other surface improvements is common and not associated with earthquake damage (photo credit: Exponent).

- **Swimming pools.** Swimming pools can be an effective means by which to assess the levelness of at-grade improvements (Figure 6-10). Typically, swimming pool waterline tiles are installed reasonably level and significant deviations from level may be indications of earth movement, earthquake-induced or otherwise. Fresh-appearing out-of-levelness of the pool coping inconsistent with construction tolerances and fresh appearing cracking of the pool shell are indicators of potential earthquake-induced permanent ground deformation. Chemical residues, if present on the waterline tile, can indicate the pre-earthquake water surface location. The degree to which these residues form a non-level surface may indicate if swimming pool deformations occurred prior to or as a result of the earthquake.

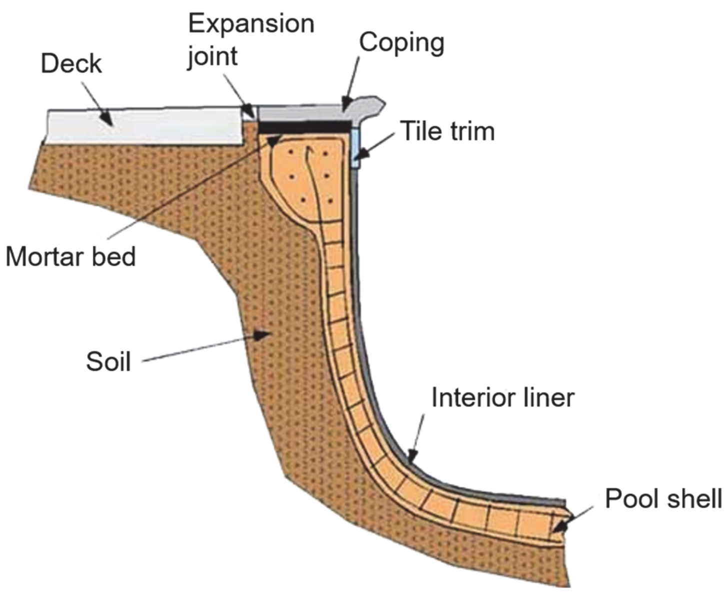


Figure 6-10 Schematic of typical swimming pool detail (image credit: Exponent).

- **Foundations.** Indicators of earthquake-induced permanent ground deformation related to foundation performance include: (a) fresh cracking in concrete foundation elements; (b) soil deformations adjacent to foundations (e.g., bulging or tension cracks); and (c) out-of-levelness consistent with conspicuous earthquake damage to adjacent house walls (Figure 6-11).
    - **Fresh cracking in concrete foundation elements** requires direct inspection of the foundation elements. This inspection can be performed along the sides of a structure, from direct inspection of interior floor slabs (which may require rolling back carpet), or from crawlspace inspections. If foundation concrete is not directly visible, geotechnical consultants can look for fresh cracks in tiles applied to slab-on-grade surfaces or in wall finishes (e.g., stucco) above the foundation elements. Fresh cracking in any of these elements indicates potential earthquake-induced permanent ground deformation. Conversely, patching of cracks, debris within cracks, or evidence of previous releveling are indicators of long-term, re-occurring ground movement unrelated to earthquake shaking (Figure 6-12, Figure 6-13, and Figure 6-14).

Figure 6-11 Foundation and ground cracking following the 2003 San Simeon, California earthquake, where the photo on the right shows the offset present across foundation crack (photo credits: Exponent).

Figure 6-12 Grout completely filling the slab-on-grade crack (indicated by pencil) indicates that slab crack was present at the time of the tile installation and has not experienced movement since tile installation (photo credit: Exponent).

Figure 6-13 Paint within concrete block wall crack indicates that the crack was present at the time of last painting (photo credit: Exponent).

Figure 6-14 Dirt and efflorescence indicates age of foundation wall crack is not recent (photo credit: Exponent).

- **Soil deformation near the foundation** involves permanent ground deformation that is localized around foundations, indicating full or partial bearing failure or foundation sliding (Figure 6-15). This mode of permanent ground deformation usually results from seismic pore-pressure induced strength loss in foundation soils (i.e., liquefaction), and in past earthquakes has generally only been observed in structures with significant bearing loads (typically 3 stories or more in height).

Figure 6-15 Ground separation from residence along slope side of structure following the 2003 San Simeon, California earthquake (image credit: Exponent).

- **Out-of-level floors** are best assessed with floor-level surveys along with inspection of adjoining wall surfaces. Floors that are significantly out of level as a result of earthquake-induced permanent ground deformation will generally have a consistent pattern of wall finish damage. Conversely, out-of-level floors accompanied by undamaged walls suggests that the out-of-level condition likely pre-existed the earthquake. Finally, out-of-level floors accompanied by damaged walls with a history of interior finish patching indicates the presence of long-term, re-occurring ground movement (e.g., slope creep or expansive soil), indicators of which may or may not have been worsened by the earthquake.

Cracking and out-of-level floors are present in virtually every foundation. Accordingly, care should be exercised to distinguish potential earthquake-induced permanent ground deformation damage from pre-earthquake conditions. In some cases, the distinction will not be clear, particularly if reconnaissance is performed many months following the earthquake. In such cases, further analysis may be necessary to determine the likely cause of the observed damage. Procedures for performing such analyses are described in Chapter 7.

- **House finishes.** Finish elements that should be inspected and documented include wall finishes (e.g., stucco), masonry block walls, and chimneys. All of these elements tend to be vulnerable to damage from earthquake shaking. Hence, the presence of damage to such elements is not necessarily indicative of permanent ground deformation. To assess the potential contribution of permanent ground deformation to damage in these elements, a pattern of ground deformations should be present based on inspections of other components of the site, as described above. For example, if only minor damage to finish elements is observed at a site with no other corroborating evidence of ground deformations, it is unlikely that earthquake-induced permanent ground deformation occurred. More information on inspecting a house for structural earthquake damage is provided in Chapter 2 and Chapter 3.

Once damage to the above elements has been documented, the geotechnical consultant should look for patterns in the site performance (e.g., cracking) to evaluate whether the damage is indicative of a large permanent ground deformation mechanism (e.g., slope failure, differential settlement across a fill pad) or is a localized phenomenon (e.g., localized settlement). Random cracking that does not form a pattern is likely not associated with permanent ground deformation. If detailed reconnaissance performed shortly after the earthquake does not reveal evidence of ground deformation, then it can be concluded that no mechanism of seismic-induced permanent ground deformation occurred at the site and therefore could not be responsible for damage to improvements.

If clear evidence of seismic permanent ground deformation is found, the final objective of the reconnaissance is to identify additional investigation activities (i.e., geotechnical subsurface investigation) that may be needed to identify the specific mechanism of permanent ground deformation and to facilitate repair or mitigation measures. Similarly, if evidence of permanent ground deformation is found (e.g., settlement or cracking), but it cannot be definitively attributed to either a static or seismic damage mechanism or a single seismic damage mechanism, the geotechnical consultant should identify additional investigation activities that may be needed to discern the mechanism of damage.

6.4 Methods of Site Reconnaissance and Investigation

The following sections discuss site reconnaissance and investigation methods specific to the ground deformation mechanisms introduced in Chapter 5.

6.4.1 Surface Fault Rupture

An investigation into whether surface fault rupture is a viable mechanism of permanent ground deformation for a given site typically begins with a review of relevant literature and digital resources for the earthquake in question. Surface faulting is generally carefully mapped by scientists and engineers specializing in the subject, and this information is publicly distributed through organizations such as the USGS. This information is often available relatively shortly after an earthquake. The results of such investigations are generally adequate to evaluate if principal faulting occurred at or near a site. To evaluate if distributed faulting occurred at a site, detailed geologic studies of the site in question and its surrounding area generally are required.

The geotechnical consultant should look for ground cracking and vertical and horizontal ground displacements (see Table 5-1 and Section 5.2). If permanent ground deformations indicating surface fault rupture are observed at the site, the geotechnical consultant should look for regional ground deformation patterns consistent with surface fault rupture to confirm that the observations are not attributable to another form of permanent ground deformation, such as landsliding.

Trenching through ground cracks can also be useful to gain insight into the source of the cracks. Fieldwork of this type is non-trivial, and guidelines for such investigations are presented by the California Geological Survey (2002). It is strongly recommended that such work be performed by geologists or engineering geologists experienced in fault mapping.

If detailed reconnaissance performed shortly after the earthquake does not reveal evidence of ground deformation, then it can be concluded that neither surface rupture nor another mechanism of earthquake-induced permanent ground deformation occurred at the site.

If data from site reconnaissance performed shortly after the earthquake are not available, and the site contains evidence of recent ground deformation, a site-specific investigation of surface fault rupture can at best provide the following information:

- Proximity of the site to the nearest mapped surface rupture features from publicly distributed mapping products, which can be used to assess the potential for principal faulting at the site. Locations of primary surface rupture generally will be available in the scientific and engineering literature within several seeks of a major earthquake.
- Probability of distributed surface rupture occurring at a particular distance from the principal rupture, which can be used to estimate the potential for distributed faulting at the site in a general sense. A discussion of analysis methods for evaluation of distributed surface fault rupture is provided in Chapter 7.

6.4.2 Liquefaction

An investigation into whether liquefaction is a viable mechanism of permanent ground deformation for a given site begins with a thorough reconnaissance of the site and the surrounding region.

The geotechnical consultant should look for evidence of liquefaction at the site, such as sand boils, ground cracking patterns consistent with ground oscillation or lateral spreading, indicators of recent foundation settlement in the structure, and foundation settlement relative to surrounding ground (see Table 5-1and Section 5.3). For structures supported on shallow foundations, liquefaction effects can include foundation bearing failures, foundation settlement or tilting or both, and lateral translations of foundations. Depending on their stiffness and strength and the magnitude of deformations, the foundation elements themselves may or may not be structurally damaged as these deformations occur.

Additional information that may be useful to the evaluation of whether liquefaction may have occurred at the site or surrounding area includes the review of relevant literature. Post-earthquake reconnaissance reports, such as by GEER, will typically document locations where liquefaction was and was not observed. The USGS publishes liquefaction maps for earthquakes shortly after their occurrence that are useful to review prior to performing a reconnaissance of a property. The maps provide initial awareness of the overall extent and importance of potential liquefaction and indicate areas in which they are most likely to have occurred. The maps are the result of liquefaction models that provide regional estimates of liquefaction hazard triggered by an earthquake. While the maps do not predict specific occurrences, they provide useful information that will inform a reconnaissance. Other literature that should be reviewed includes liquefaction hazard maps prepared by state geological surveys and regional compilations of surface geological and groundwater data.

If detailed reconnaissance performed shortly after the earthquake does not reveal evidence of ground deformation, it is likely that liquefaction phenomena either did not occur at the site or liquefaction occurred at sufficiently large depths that its effects were not manifest at the surface. However, if such field reconnaissance data are not available and the site contains evidence of recent ground deformation, additional analysis may be required to evaluate the potential for liquefaction. If surface evidence indicates liquefaction may have occurred but is not a conclusive indicator, a screening analysis may be performed to investigate liquefaction potential at the site. The screening analysis is described in Chapter 7. If the screening analysis indicates that the site may be susceptible to liquefaction, more detailed investigations involving subsurface exploration, laboratory testing, and engineering analyses typically will need to be performed to evaluate whether liquefaction was likely to have been triggered and its effects. These are described in Chapter 7.

6.4.3 Seismic Compression

An investigation into whether seismic compression is a viable mechanism of permanent ground deformation for a given site begins with a thorough site reconnaissance. Seismic compression tends to be a localized phenomenon, and hence the reconnaissance activities should be focused on the site in question and the immediately surrounding area.

The geotechnical consultant should look for local ground deformation patterns, such as those described in Section 6.3 (see also Table 5-1 and Section 5.4). Reconnaissance should be performed as soon as possible after the earthquake, so that potential cracks in the soil or flatwork will be fresh and thus readily distinguishable from cracks that may have pre-existed the earthquake.

If present, the lateral extent and depth of fill materials should be mapped based on available information, which might include geomorphic observations, aerial photos, or as-built grading plans for the site. Ground deformations should be interpreted relative to the fill section geometry to evaluate whether there is an association of ground cracks with the presence of fill. The lack of such association would suggest that seismic compression was unlikely to have been the cause of cracks.

If detailed reconnaissance performed shortly after the earthquake does not reveal evidence of ground deformation, it is not likely that seismic compression resulted in damaging ground displacements at the site. However, if such field reconnaissance data are not available and the site contains evidence of recent ground deformation, detailed investigations may be necessary to investigate the likelihood that seismic compression occurred at the site. These investigations involve subsurface exploration, laboratory testing, and engineering analyses (see Chapter 7).

6.4.4 Landsliding

An investigation into whether earthquake-induced landsliding is a viable mechanism of permanent ground deformation for a given site begins with a thorough reconnaissance of the site in question as well as the surrounding region.

Additional information that may be useful to the evaluation of whether landsliding may have occurred at the site or surrounding area includes the review of relevant literature. The USGS publishes landslide maps for earthquakes shortly after their occurrence that are useful to review prior to performing a reconnaissance of a property. The maps provide initial awareness of the overall extent and importance of potential landsliding and indicate areas in which they are most likely to have occurred. The maps are the result of landslide models that provide regional estimates of landslide hazard triggered by an earthquake. While the maps do not predict specific occurrences, they provide useful information that will inform a reconnaissance.

The geotechnical consultant should look for regional ground deformation patterns, such as extensional ground cracking in potential scarp areas and compressive bulging of the slope near the toe (see Table 5-1 and Section 5.5). Extensional ground cracking with no observed slide may indicate an existing instability. Caution should be exercised when conducting reconnaissance on the down-slope side of observed tension cracks and areas that have already experienced movement.

The local geology should be mapped by an experienced engineering geologist, with particular attention paid to the orientation of bedding planes relative to the slope in question (a formal report by a certified engineering geologist may be required by local municipalities as part of a redevelopment process and may be on file with the municipality and available for review). Bedding planes that intersect a slope or that are subparallel to a slope should be noted. The geologic investigation should also include a careful study of aerial photographs to investigate the presence of geomorphic features that are consistent with past landsliding. Ground deformations should be interpreted relative to the geology to evaluate whether the landslide slip plane is along bedding or crosses the bedding planes.

If detailed reconnaissance performed shortly after the earthquake does not reveal evidence of ground deformation, it is not likely that landslide phenomena occurred at the site. However, if such field reconnaissance data are not available and the site contains evidence of recent ground deformation, detailed analytical studies may be needed to investigate the potential for landslide development during the earthquake. Those studies entail subsurface exploration, laboratory testing, and engineering analyses (see Chapter 7).

6.4.5 Retaining Wall Deformation

The objectives of site reconnaissance for retaining wall studies are to: (1) examine the backfill soil and the ground below the toe of the wall for evidence of wall instability in sliding, tilting, or bearing capacity; and (2) examine the wall for structural damage (e.g., cracking at the base of the stem of a cantilever wall).

The geotechnical consultant should look for evidence of wall rotation, sliding, cracking, separation between the wall and backfill, bulging of wall face (reinforced earth wall), loss of wall components (reinforced earth wall) and flatwork cracking behind the retaining wall that may suggest either wall movement or permanent ground deformation (see Table 5-1 and Section 5.6).

Observed wall movement and distress should be carefully examined to distinguish between earthquake-induced wall movement and long-term wall movement. Note that narrow (i.e., less than 1/8-inch-wide) vertical cracks without significant out-of-plane offsets are generally related to concrete shrinkage and not indicative of earthquake-induced damage to the wall. If detailed reconnaissance performed shortly after the earthquake does not reveal evidence of ground deformation, earthquake-induced wall movement, or fresh appearing cracking of the wall, it is likely that earthquake-induced retaining wall damage did not occur at the site, or earthquake-induced wall movement occurred at such a small magnitude that it is not manifest in the wall and nearby improvements and therefore did not damage the wall.

Retaining walls containing structural elements (e.g., cantilever retaining walls) should be examined for potential occurrence of structural damage and wall rotation. Geotechnical consultants should measure the wall rotation at regularly spaced intervals. For walls where the rotation exceeds levels required to mobilize active and passive earth pressures (Figure 6-16), the geotechnical consultant should examine: (1) the base of the stem at the footing for horizontal cracking consistent with stem bending; (2) the angle between the stem and base for stem bending; and (3) the levelness of the base for indications of overall tilting.

Detailed investigations involving subsurface exploration, laboratory testing, and engineering analyses may need to be performed when field indicators of potential earthquake-induced wall movement are present (e.g., rotation in excess of that required to mobilize active and passive earth pressures as delineated in Figure 6-16), unusual cracking of the wall is observed, or distress to the flatwork behind the wall is present. The purpose of these investigations is to identify the cause of the retaining wall damage and to facilitate the design of repairs. Detailed investigations and analysis may also be required to help distinguish between wall damage caused by long-term static processes and seismic loading (see Chapter 7).

SOIL TYPE AND CONDITION	ROTATION, Y/H*	
	ACTIVE	PASSIVE
Dense cohesionless	0.001	0.02
Loose cohesionless	0.004	0.06
Stiff cohesive	0.010	0.02
Soft cohesive	0.020	0.04

* Y = horizontal displacement and H = height of wall

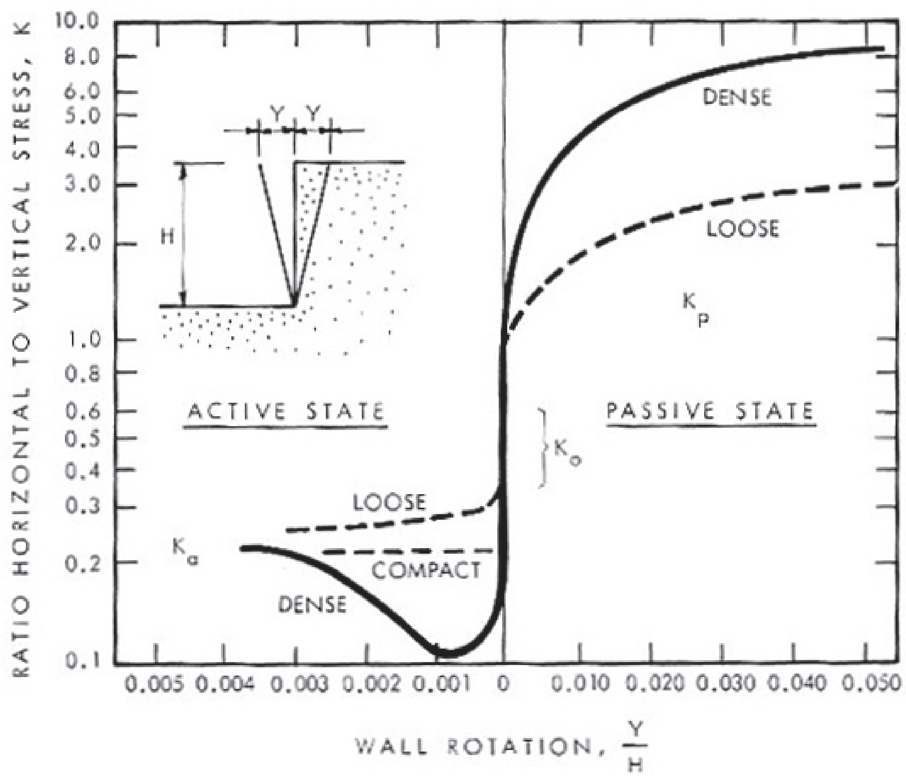


Figure 6-16 Magnitude of wall rotation to reach failure in retained soil and effect of wall rotation on earth pressures. K_o, K_a, and K_p are the at-rest, active, and passive earth pressure coefficients, respectively (Canadian Geotechnical Society, 1992).

CHAPTER 7

Damage Repair: Geotechnical

7.1 Introduction

This chapter addresses evaluation of and conceptual repair and mitigation strategies for earthquake-induced permanent ground deformation. The geotechnical consultant should utilize the analysis tools and repair and mitigation strategies discussed herein after the completion of the site damage investigation described in Chapter 6. The overall objective of the repairs recommended in the *Engineering Guidelines* is to restore the house to its pre-earthquake condition. When earthquake-induced permanent ground deformation is determined to have caused damage to a house or associated surface improvements, the *Engineering Guidelines* provide guidance on the evaluation and identification of repair and mitigation strategies to address the earthquake-induced permanent ground deformation, so that the house and surface improvements can be restored to their pre-earthquake conditions. Appropriate repairs may address only damage to the house and surface improvements, or may include mitigation measures intended to lessen the severity of the consequences of the mechanism causing the earthquake-induced permanent ground deformation.

Prior to making repair and mitigation recommendations, the geotechnical consultant should have a clear understanding of site conditions and the ground deformation mechanisms causing the observed damage at the site. If the underlying damage mechanisms are unclear, additional investigation and analyses may be required before proceeding with repair and mitigation recommendations. (Section 6.3 describes the conditions under which geotechnical investigations involving subsurface exploration, laboratory testing, and engineering analyses are typically needed.) This chapter presumes that the geotechnical consultant has experience applying the described repair and mitigation techniques and has knowledge about their limitations, risks, costs, and appropriateness for specific site conditions.

Following an earthquake, questions regarding soil “damage” and its repair frequently emerge. In this context, soil “damage” is defined as an earthquake-induced disruption of the soil that reduces the ability of the soil to support imposed loading from improvements. In this chapter, soil damage and guidance on evaluation and repair and mitigation strategies are discussed for each deformation mechanism described in Chapter 5 and Chapter 6: surface fault rupture, liquefaction, seismic compression, landsliding, and retaining wall deformation.

7.2 Surface Fault Rupture

Based on a post-earthquake site investigation and a review of the mapped surface fault trace, identification of surface fault rupture at a site will typically be straightforward. Maps of fault traces can be obtained from the USGS, the California Geological Survey, and other state geological surveys.

If a site is located near but not in the immediate vicinity of the surface expression of the fault trace (i.e., ground cracks are unlikely to be due to principal faulting), analysis can be performed to evaluate if observed ground deformations are potentially related to distributed surface rupture. Appendix E contains a short description of this analysis.

7.2.1 Soil Damage

Within the context of surface fault rupture, geologic material damage could be said to occur if earthquake-induced displacement across a discontinuity increased the likelihood of future fault-induced displacements across the same discontinuity, or if the displacements increase the likelihood of other modes of permanent ground deformation, such as landsliding involving deformations across the discontinuity. Earthquakes are an ongoing process on the geologic time scale and hence have occurred repeatedly in the past on faults and will continue in the future. Accordingly, the occurrence of an earthquake and its associated surface rupture in the present time is not a unique event; it likely has occurred before on the same fault. Therefore, there is no basis to assume that the occurrence of surface rupture across pre-existing discontinuities has increased the likelihood for future displacements across those same discontinuities.

The only case where geologic material damage could possibly occur is if a discontinuity is identified at a site that was not present prior to the earthquake and displacements occurred across the discontinuity from the earthquake (this situation is expected to be very unusual). The discontinuity could experience additional displacements in future events, and hence by the above definition, the geologic materials at the site could be said to have been damaged. Moreover, the displacements across the discontinuity would likely have reduced the strength of the geologic materials, possibly as low as residual strength, which could affect the potential for future landslides, particularly if the discontinuity is adversely oriented with respect to the surface topography.

7.2.2 Repair and Mitigation Strategies

After an earthquake that has caused (or is suspected to have caused) fault rupture at a site, there are several significant engineering issues that affect the development of an appropriate repair plan. First, did the occurrence of fault rupture damage the soil or a structure at the ground surface? Second, what are appropriate repair and mitigation strategies for the property? Prior to making repair and mitigation recommendations, the geotechnical consultant should have a clear understanding of site conditions, including the amount of vertical and horizontal fault rupture deformations at the property, the damage caused by those deformations, and the potential for future fault rupture at the property.

The repair and mitigation measures recommended by the geotechnical consultant should satisfy two criteria. First, the recommendations should be appropriate and consistent with the magnitude of ground deformation and collateral physical damage observed at the property. Second, the recommendations should be consistent with an analysis of the potential fault rupture hazards that may still exist at the property. Appropriate repairs may address only damage to the house and surface improvements, or may include mitigation measures as well.

Buildings affected by surface fault rupture are often damaged so severely that repair is not practical. The feasibility of rebuilding on a site affected by surface fault rupture will depend upon applicable laws and the disruption to the site topography. Where repair is practical and allowed by law (e.g., in areas of minor surface disruption, such as areas of distributed surface rupture), the method and extent of building repair depends on the nature, extent, and significance of the damage and, for foundation repair, is usually determined by the structural technical consultant, incorporating recommendations provided by the geotechnical consultant (see Section 4.2 and Section 4.3). Except as discussed in the remainder of this section, the future stability of a site is generally not affected by surface fault rupture.

Bray (2001) has presented three mitigation strategies for hazards associated with future surface fault rupture, summarized below. These strategies are generally considered in the context of new construction as opposed to repair of a damaged building. Each strategy requires proper interpretation of the geology on both a regional and site-specific basis. The geotechnical consultant is referred to Bray (2001) for a full description of each strategy if the methodology is to be implemented.

- The first strategy is avoidance, or more specifically, setting a structure back a particular distance from the primary fault trace. This approach is the rationale of the State of California Alquist-Priolo Act, which requires the mapping of known, active fault traces and prohibits new construction within 50 feet of the fault. This strategy may not always be practical for residential sites because of limited lot size. Moreover, localized displacements from distributed rupture may not be avoidable.
- The second strategy is to "absorb" fault displacements within ductile blankets of soil, which might include compacted fill soils reinforced with geosynthetics. These ductile soil blankets distribute ground deformations across a relatively wide zone and hence may be useful for mitigating displacements across discontinuities (from primary or distributed surface rupture).
- The third strategy is to increase the deformation tolerance of foundations for structures. This generally involves constructing a foundation system with lateral continuity (i.e., a mat foundation or footings inter-connected with grade beams) and significant reinforcement to ensure structural strength and ductility. For strike-slip ruptures, the construction of shallow foundations atop a double layer of smooth plastic sheets sandwiched between clean sands—defined in ASTM D 2487 as those with fines content less than 5%—or fine gravels can help mitigate the transfer of tensile ground strains to the foundation. Deep foundation elements, such as driven piles or drilled shafts, should generally be avoided in potential surface rupture zones, because these foundations do not allow decoupling of ground displacements from foundation displacements.

7.3 Liquefaction

If geotechnical damage indicates potential, but not definitive, liquefaction, analysis may be necessary to determine the causative mechanism of the observed damage and proceed with developing a repair strategy (e.g., settlement and cracks *without* ejecta may indicate liquefaction or seismic compression, whereas settlement and cracks *with* ejecta definitively indicate liquefaction and potentially seismic compression as well). Liquefaction analyses presented herein fall into two categories: screening analysis and detailed analysis.

The purpose of a screening analysis is to determine if a site has obvious characteristics that would indicate that liquefaction could not have occurred. If the screening investigation clearly indicates the absence of liquefaction susceptibility, further analysis is not required of the site for liquefaction. When the soil at a site is judged to be potentially susceptible to liquefaction based on screening analysis procedures, more detailed analyses may be needed to evaluate whether liquefaction was likely or unlikely to have actually occurred. The framework for liquefaction assessment is essentially an assessment of: (1) liquefaction triggering; and (2) liquefaction consequences. Appendix F contains detailed descriptions of the screening and liquefaction analyses.

7.3.1 Soil Damage

The effect of liquefaction and post-liquefaction reconsolidation of the soil itself is to slightly densify the material. This would slightly increase the static shear strength of the soil relative to the pre-earthquake condition and therefore does not qualify as soil damage. However, the potential ground deformations resulting from liquefaction may require repair and mitigation.

7.3.2 Repair and Mitigation Strategies

In the aftermath of an earthquake that has caused (or is suspected to have caused) liquefaction at a site, there are several significant engineering issues that affect the development of an appropriate repair plan. First, what are appropriate repair and mitigation strategies for the property? Second, did the occurrence of liquefaction damage the soil or a structure at the ground surface? Prior to making repair and mitigation recommendations, a geotechnical consultant should have a clear understanding of site conditions, including the amount of vertical and horizontal liquefaction-induced deformations at the property, the damage caused by those deformations, the depth and lateral extent of liquefiable soils at the property, and the potential future liquefaction hazards at the property.

The repair and mitigation measures recommended by the geotechnical consultant should satisfy two criteria. First, the recommendations should be appropriate and consistent with the magnitude of ground deformation and collateral physical damage observed at the property. Second, the recommendations should be consistent with an analysis (elaborated in more detail in CDMG, 1997) of the potential deformations and liquefaction hazards that may still exist at the property. Appropriate repairs may address only damage to the house and surface improvements, or may include mitigation measures as well.

For sites with minor or moderate liquefaction-induced damage or for sites where mitigation measures are not practical, the most appropriate repair is often to repair the damaged improvements and accept the risk of potential liquefaction in future events. Such repair strategies may include releveling of the structure on the foundation, releveling of the foundation using available techniques (e.g., pressure grouting), or releveling and repair of fractured foundations. See Section 4.3 for a discussion of structural repair of foundations.

In cases of severe liquefaction or where mitigation of the liquefaction hazard is desired for improved future performance, structural modification or ground improvement or both may be considered. CDMG (1997) and Martin and Lew (1999) contain a detailed discussion of these options along with references.

Structural modification options typically do not address the soil's susceptibility for liquefaction-induced ground deformations but seek to mitigate the effects of these deformations on the structure. The method and extent of the structural options inherently depends on the type of liquefaction-induced ground movement (i.e., amount of lateral movement, amount of settlement, degree of differential ground displacements). Under certain conditions, reinforcing or interconnecting existing foundations is feasible. Under other conditions, construction of new foundations—such as post-tensioned slabs, mat foundations, piles, or caissons—is required.

Ground improvement options seek to either eliminate or reduce the soil's susceptibility for liquefaction-induced ground deformations. These options do not necessarily reduce the potential for structural damage during future earthquakes from strong shaking. When the intent of ground improvement is to reduce the susceptibility for liquefaction-induced deformation, the structure should be designed to accommodate the deformations that may occur in future earthquakes. Ground improvement options include vibro-compaction, vibro-replacement, deep dynamic compaction, compaction grouting, permeation grouting, soil mixing, and jet grouting. Additional options include removal and replacement of liquefiable soils, modification of site geometry, and drainage to lower the ground water. For lateral spreads and flow failures, construction of containment structures is also an option.

It is possible to use these various options individually or in combination for a site. For example, a combination of ground improvement methods or a combination of ground improvement and structural modification may be practical.

7.4 Seismic Compression

An analysis of seismic compression for a site begins with an assessment of susceptibility. Susceptible soils include granular soils, silts, and low-plasticity clays. Two simplified procedures for estimating ground displacements from seismic compression are presented in Appendix G.

7.4.1 Soil Damage

The effect of seismic compression on the soil itself is to slightly densify the material. The extent to which seismic compression affects the potential for future hydro-compression (and vice-versa) is unknown, although an adverse effect is not expected. The occurrence of seismic compression would not be expected to adversely affect the susceptibility of the soil to shear failure (i.e., landslides). Thus, measures to improve the soil following the occurrence of seismic compression are generally not warranted.

7.4.2 Repair and Mitigation Strategies

In the aftermath of an earthquake that has caused (or is suspected to have caused) volumetric strain-induced settlement at a site, there are several significant engineering issues that affect the development of an appropriate repair plan. First, did the occurrence of seismic compression damage the soil or a structure at the ground surface? Second, what are appropriate repair and mitigation strategies for the property? In this context, soil damage is defined as an earthquake-induced disruption of the soil that reduces the capacity of the soil to reliably support the imposed loads of improvements at the property.

Prior to making repair and mitigation recommendations, a geotechnical consultant should have a clear understanding of site conditions, including the depth and lateral extent of soils at the property that have undergone volumetric strain, the amount of vertical and horizontal volumetric strain-induced deformations at the property, the damage caused by those deformations, the depth and lateral extent of "loose" soils at the property, and the potential future volumetric strain-induced movement at the property.

The repair and mitigation measures recommended by the geotechnical consultant should satisfy two criteria. First, the recommendations should be appropriate and consistent with the magnitude of ground deformation and collateral physical damage observed at the property. Second, the recommendations should be consistent with an analysis of the deformation potential that may still exist at the property. Appropriate repairs may address only damage to the house and surface improvements, or may include mitigation measures as well.

Ground displacements from seismic compression may damage structures by inducing differential settlements and local ground extension. Repair strategies may include: (1) releveling of the structure on the foundation; (2) releveling, repair, and underpinning of the foundation; and (3) removal and replacement with a new foundation, such as a relatively strong and stiff shallow foundations or drilled shafts inter-connected by grade beams. See Section 4.3 for a discussion of structural repair of foundations.

7.5 Landsliding

An analysis of seismic slope stability considers the following potential site conditions:

1. sites where the earthquake is likely to significantly weaken the slope material
2. sites that have weakly cemented rock or soils
3. sites comprised of materials whose strength is unlikely to be significantly compromised by the earthquake

Analyses specific to each set of site conditions are described in Appendix H and provide insight into the likelihood of a seismically induced landslide in the case of conditions 1 and 2 or the magnitude of slope deformation in the case of condition 3.

7.5.1 Soil Damage

If the site under consideration has no history of landslide activity (including no ancient landslides) but experienced a landslide as a result of an earthquake, the slope materials may have become damaged along the landslide slip surface as a result of the earthquake. This damage would be associated with the development of a residual strength condition along the slip surface that was not present prior to the earthquake. The development of an essentially permanent residual strength condition only occurs in relatively clayey soil or clayey rock materials and occurs because of clay fabric reorientation, such that clay particles are aligned with the direction of slip. This phenomenon would not be expected in nonplastic sands, silts, gravels, or bedrock composed of those material types because of the lack of platy particle shapes. Another circumstance under which soil damage could occur is in cemented soil or rock materials in which the peak strength is significantly higher than the residual strength due to the presence of the cementing agent. De-aggregation of the material due to earthquake-induced deformations could be considered soil damage. Soil damage under either of these scenarios is only significant with respect to the future performance of the site if the stability of the post-earthquake slope (potentially with residual strengths) is reduced from the pre-earthquake stability (with the appropriate pre-earthquake strengths).

In either of these cases, a slope that is judged to be significantly "damaged" soil could be "repaired" through removal and replacement with properly engineered compacted fill or with the application of appropriate in situ soil improvement techniques.

If a site has no evidence of earthquake-induced ground deformation or if deformations occurred under conditions different from those described above (e.g., raveling of a fill slope or reactivation of a pre-existing landslide), the slope material's resistance to landsliding would not be expected to have been compromised as a result of the earthquake.

7.5.2 Repair and Mitigation Strategies

In the aftermath of an earthquake that has caused (or is suspected to have caused) slope failure at a site, there are several significant engineering issues that affect the development of an appropriate repair plan. First, did the occurrence of landsliding damage the soil or a structure at the ground surface? Second, what are appropriate repair and mitigation strategies for the property?

Prior to making repair and mitigation recommendations, the geotechnical consultant should have a clear understanding of site conditions, including the amount of vertical and horizontal earthquake-induced deformations at the property, the damage caused by those deformations, the depth and extent of failure surface, the static stability of the slide mass, and the potential future landslide movement at the property.

The repair and mitigation measures recommended by the geotechnical consultant should satisfy two criteria. First, the recommendations should be appropriate and consistent with the magnitude of ground movement and collateral physical damage observed at the property. Second, the recommendations should be consistent with the potential for ongoing or future movement of the slide mass. Appropriate repairs may address only damage to the house and surface improvements, or may include mitigation measures as well.

Where analysis indicates satisfactory stability in the absence of strong ground shaking (i.e., lurching) and mitigation is not desired, the appropriate course of action is repair of damaged improvements. Where a structure has been impacted by a slide of limited extent, support of the foundation may be restored by underpinning extending through the slide mass and designed to withstand future soil movements.

Where analysis indicates a high potential for continued or future movement under static conditions, mitigation of slide movement is necessary prior to repair of damaged improvements. The most common methods of landslide hazard mitigation are: (1) grading to improve the slope stability; (2) reinforcement of the slope or improvement of the soil within the slope to enhance the slope stability; (3) dewatering to improve the slope stability; and (4) reinforcement of the affected foundations so that they are more able to tolerate anticipated future displacements. If practical mitigation techniques are inadequate to mitigate the landslide hazard, it may be necessary to abandon the site. See Section 4.3 for a discussion of structural repair of foundations.

Common grading options for improving slope stability include flattening the slope or decreasing its height, excavation of the slide mass and replacement with compacted fill, and the placement of fill against a slope face to buttress the slope.

Common reinforcement options for slopes include the use of deep foundations, such as piles or drilled shafts, installation of tieback anchors or soil nails, and the installation of retaining walls. Soil improvement techniques include in situ soil mixing with lime and cement. Techniques for in situ soil densification can also be effective in liquefiable soils (see Section 7.3.2 for details).

Dewatering can be a very effective mitigation strategy, but the de-watering system should be maintained over the life of the project. Dewatering includes the efficient removal of surface water with surface drains, as well as the collection and removal of subsurface water with subdrains or wells.

Reinforcement of foundations or modifying the foundation configuration can enhance the resistance of structures to slope deformations. For example, drilled shafts and grade beams can be installed in lieu of shallow foundations. Alternatively, shallow unreinforced or weakly reinforced foundations can be replaced or supplemented with well-reinforced and interconnected footings or a mat foundation, although such systems may still be subject to tilt in the event of future landslide movements.

Additional details on mitigation and repair strategies are presented in Chapter 12 of Blake et al. (2002).

7.6 Retaining Wall Deformation

A common approach to analysis of retaining walls for design is extending static limit equilibrium analysis to pseudo-static conditions and checking if equilibrium is satisfied. These approaches have been applied to yielding and non-yielding retaining walls and are discussed in Appendix I.

7.6.1 Soil Damage

Earthquake-induced retaining wall movements are typically associated with backfill soils reaching their peak strength and the formation of a passive wedge below the toe of the wall. As discussed in Appendix I, interpretation of calculated pseudo-static factors of safety and calculated seismic wall displacements should occur in the context of the condition of the wall, potential (small) design displacements that may have already occurred, and the collateral soil stresses induced by the static and seismic wall movement. When earthquake-induced wall movements are small, and analyses of future wall stability (described in Appendix I) indicate that the wall is adequately stable or serviceable, soil damage is not possible.

If the wall and backfill material under consideration have no history of creep or movement but experience enough movement along a slip surface in the backfill as a result of an earthquake, the backfill materials may have lost strength along the slip surface as a result of the earthquake. This damage would be associated with the development of a residual strength condition along the slip surface that was not present prior to the earthquake. The development of an essentially permanent residual strength condition only occurs in relatively clayey soil or clayey rock materials, and occurs because of clay fabric reorientation, such that clay particles are aligned with the direction of slip. This phenomenon would not be expected in nonplastic sands, silts, gravels, or rock composed of those material types because of the lack of platy particle shapes. The “damaged” soil could be “repaired” through removal and replacement with properly engineered compacted fill or with the application of appropriate in situ soil improvement techniques. Another option is to structurally increase the wall’s capacity to support the increased soil loads that have resulted from the loss of soil strength.

If a site has no evidence of earthquake-induced ground deformation, or if deformations occurred under conditions different from those described above, the material’s ability to resist retaining wall movement would not be expected to have been compromised as a result of the earthquake.

7.6.2 Repair and Mitigation Strategies

In the aftermath of an earthquake that has caused (or is suspected to have caused) retaining wall movement or failure at a site, there are several significant engineering issues that affect the development of an appropriate repair plan. First, did the occurrence of retaining wall movement or failure damage the soil or the retaining structure itself? Second, what are appropriate repair and mitigation strategies for the property?

Prior to making repair and mitigation recommendations, the geotechnical consultant should have a clear understanding of site conditions, including the mode of movement of the wall and the engineering properties of the soils supporting and supported by the wall, and the relationship between the retaining wall and other improvements on the site.

The repair and mitigation measures recommended by the geotechnical consultant should satisfy two criteria. First, the recommendations should be appropriate and consistent with the nature and extent of the retaining wall deformation. Second, the recommendations should be consistent with an analysis of the potential future movement of the wall and the consequences of that movement. Appropriate repairs may address only damage to the retaining wall and improvements supported by the retaining wall or may include mitigation measures as well.

Retaining wall damage is defined as deformation that has changed the stability below minimum requirements under reasonable loading, altered the serviceability, or affected the appearance of the wall. In the latter instance, where the fundamental structural condition of the wall has not been altered, repairs to the wall are cosmetic and are not discussed in this section. For example, cracking of a concrete retaining wall that has not fundamentally altered the structural condition of the structure can be epoxy injected to seal the cracks and impede reinforcement corrosion.

In order to determine if damage has occurred to a wall, the following engineering analyses should be performed: (1) analysis of the wall system stability; (2) evaluation of the structural condition of the wall; and (3) integrity of the drainage system.

Analysis of the wall system stability can be performed to evaluate factors of safety against sliding, overturning, and bearing capacity. These analyses should utilize soil strengths appropriate for the post-seismic condition (i.e., residual strengths should be used if significant shear strains in backfill or foundation materials was likely—see discussion in previous section) and reasonable surcharge loading and drainage conditions that may be expected for the remainder of the wall's service life. These analyses, which may be performed in collaboration with a structural consultant, follow typical design methodologies (ASCE, 1994a). The results should be compared to minimum design factors of safety (see Appendix I). If the results of the analyses indicate that the wall is not adequately stable or serviceable, the technical consultant should consider repair alternatives to strengthen the wall and return it to a serviceable condition. Absent economic repair alternatives, the wall may need to be removed and replaced.

Evaluation of the structural condition of the wall should consider cracking or other distress to the wall system. Concrete cracks may be epoxy injected to restore the strength of the concrete and for the protection of the reinforcement with access to only one side of the wall for the repair. It has been shown that with appropriate selection and preparation of epoxy viscosity, proper mixing of components, and proper execution of the injection, crack repairs made from one side only of concrete elements have strengths comparable to the uncracked strengths of concrete specimens that are free of restrained shrinkage stresses (NAHB, 2002).

Drainage systems behind retaining walls may be damaged by earthquake-induced wall movements. When this occurs, the stability of the wall immediately following the earthquake may be adequate. However, if the drainage conditions are not repaired, the stability of the wall may eventually be compromised as hydrostatic pressures develop behind the wall. If the backfill soils are clays (susceptible to creep), the moisture increase in the backfill can significantly reduce the soil strength (Terzaghi et al., 1996) and increase lateral pressures exerted on the wall by the backfill soils. Evaluation of the drainage system integrity involves direct observation of drainage pipes and their connections to suitable discharge facilities via test pits or video survey. If the drainage system is damaged, it either should be repaired or the stability of the wall in the presence of high hydrostatic water pressures should be demonstrated with suitable analysis.

7.6.3 Reinforced Soil Walls and Slopes

For reinforced soil or mechanically stabilized earth (MSE) walls and slopes, repair and mitigation strategies are similar except that there are additional components to address, such as the condition of individual reinforcement layers and facing material. Generally, damage includes settlements at the reinforced-unreinforced juncture, face bulging, and separation between the face components and the reinforcements. If the facing material detaches from the reinforcements, a local loss of the soil in the reinforced zones may occur. In this case, the backfill material will need to be replaced and compacted before facing panels are reattached.

CHAPTER 8

Technical Consultant Reports

8.1 Introduction

Typically, the final product of a technical consultant's work will be a written report that summarizes findings and conclusions regarding earthquake damage and earthquake-induced permanent ground deformation, and provides a conceptual scope of repair. The report should be written to be useful to the end user, typically a property owner or an insurance adjuster, but also should provide sufficient technical content to permit independent peer review of the conclusions and recommendations. For the purpose of these *Engineering Guidelines*, it is assumed that the report will not be used directly as the basis for repair permit submittal to a building department.

8.2 Report Content

Where preparation of a written report is part of the technical consultant's scope of services, the specific format and content are not delineated by this chapter. Certain basic content, however, should be included to convey the findings, conclusions, and recommendations addressed by the *Engineering Guidelines* and to reflect the unique nature of a post-earthquake damage assessment as distinct from other engineering assessments, such as post-earthquake safety assessments, pre-purchase evaluations, seismic evaluations, and conceptual retrofit designs.

The following outline, which is intended only to identify content specifically addressed by the *Engineering Guidelines*, should be expanded or abbreviated to suit the consultant's scope of services and the needs of the report's end user. Some of the content identified below may be applicable to either the geotechnical or structural consultant only.

- Administrative information
  - Name of the property owner.
  - Address of the property.
  - Name and contact information of the party for whom the report has been prepared, if other than the property owner.
  - Insurance claim number, if applicable.
  - Name, contact information, registration stamp, and signature of the licensed professional in responsible charge of the investigation (in accordance with state licensing laws).
  - Reports produced after an earthquake often use a standardized format. Since these reports look alike, each report should use numbered pages and should include both the report date and property address (or another unique identifier) on every page.

- Scope and procedures
    - Objective of retention and scope of services (including reference to these *Engineering Guidelines* as appropriate).
    - Dates of site investigations and personnel involved.
    - Nature and extent of damage investigations, including areas not investigated (see Chapter 3 and Chapter 6).
    - Limitations and disclaimer.
- Earthquake event data
    - Ground shaking data: USGS ShakeMap (see Section 2.4) or its equivalent for the earthquake, with property location indicated. For property location, relevant values from ShakeMap (or its equivalent), such as:
        - instrumental intensity,
        - peak ground acceleration,
        - peak ground velocity, and
        - spectral acceleration.
    - Where available, published data and maps for regional post-earthquake observations of permanent ground deformation with property location indicated.
    - Summary of occupant interview, if performed.
    - Discussion regarding level of expected damage based on the historical performance of houses given the level of ground shaking at the site.
- Description of site and buildings
    - Topography of site.
    - Information regarding geologic conditions and regional and site-specific soil conditions.
    - Condition of surrounding public and private lands.
    - Description of building, including:
        - Existing tagging, if any. Post-earthquake safety assessment in accord with ATC-20-1, *Field Manual: Postearthquake Safety Evaluation of Buildings* (ATC, 2005), is outside the scope of engineering services contemplated by these guidelines; however, if in the course of the damage assessment the consultant observes dangerous conditions, the consultant should inform the client and building occupants.
        - General characterization of the building geometry and foundation type (e.g., single story slab-on-grade, split level with cripple walls, hillside house on piers).
        - Known relevant pre-existing conditions and modifications.
        - Repairs or modifications since earthquake.

- Identification of structurally significant earthquake damage and earthquake-induced permanent ground deformation, and appropriate repair and mitigation recommendations for each. Appropriate repair and mitigation recommendations are those that satisfy the objectives discussed in Chapter 4 and Chapter 7. For consistency and practicality, this section of the structural consultant's report may be organized to match the main elements of the house, as outlined in Chapter 3 and Chapter 4, and the geotechnical consultant's report may be organized as outlined in Chapter 6 and Chapter 7. Where appropriate, the report may also discuss observed patterns of non-earthquake damage and structurally or geotechnically insignificant earthquake damage, especially where resolution of those damage patterns has been requested by the client. This section of the report is commonly supplemented by one or more of the following:
    - A plan sketch of the house or site as needed to clarify the location of elements, damage observations, permanent ground deformation, and repair and mitigation recommendations discussed in the report text.
    - Identification of areas or elements that were not investigated and the reasons, such as lack of access (see Chapter 3).
    - Recommendations for additional investigation, such as destructive investigation to confirm or rule out suspected damage to concealed elements (see Chapter 3) and subsurface investigation (see Chapter 7).
    - General quantities associated with each type of repair and mitigation recommendation, if requested by the client.
    - Expected code-triggered upgrades, if any (see Appendix A); for clarity, these upgrades should be provided in a separate section than in-kind repair recommendations.
    - Recommended voluntary upgrades (see Appendix B) and soil mitigation measures intended only for improved performance in future earthquakes, if any; for clarity, these recommendations should be provided in a separate section than in-kind repair recommendations.
- Appendices
    - Damage investigation checklists and documentation (see Chapter 3 and Chapter 6).
    - Bibliography of information resources.
    - Copies of property-specific reports from subcontractors.
    - Thumbnail images of all inspection photographs.
    - Curriculum vitae or resume of technical consultant in responsible charge of work and key contributors.

APPENDIX A

Code-Triggered Upgrades

This appendix provides a brief introduction to code-triggered upgrades. When repairing earthquake damage, the governing building or residential code may allow for the house to be restored to its pre-damage condition, or code-triggered upgrades may apply. It is important that the structural consultant identify applicable code-triggered upgrades, since this additional work will usually need to occur at the time of the damage repair.

Code provisions that trigger upgrades can vary from jurisdiction to jurisdiction and can vary widely in scope. The structural consultant will first need to determine in what jurisdiction (city or county) the house is located. The structural consultant will then need to become familiar with the locally adopted codes and regulations, and confirm those codes, regulations, and their interpretation with the local building department.

Several triggers of potential significance can be found in the IEBC, where the finding of "substantial structural damage" may trigger upgrades to the gravity-load-carrying and seismic-force-resisting systems. The structural consultant is encouraged to determine if these codes are adopted in the applicable jurisdiction and if amendments are made. At the time of writing this document, the substantial structural damage triggers are not adopted for residential structures in California. Code-triggered upgrades also can include items related to mechanical, electrical, plumbing, and fire safety systems.

Examples of code-triggered upgrades that are structurally related include:

- If repair to earthquake damage is in accordance with the IEBC without modification by the local jurisdiction, it is possible for substantial structural damage to trigger a mandatory complete wind upgrade for houses. However, houses are currently exempt from seismic upgrade requirements.
- If repair to earthquake damage is in accordance with the IRC without modification by the local jurisdiction, it is possible based on interpretation of the IRC provisions that all repairs will have to conform to IRC requirements for new construction.
- If repair to earthquake damage is in accordance with either the IEBC or IRC and damage extends above certain cost of repair thresholds, flood mitigation measures may be triggered.
- Limitations on repair of wood-burning fireplaces can be set by regional air-quality regulations.

When triggers involving substantial structural damage apply, information provided in Appendix D may be of assistance in determining whether substantial structural damage has occurred.

APPENDIX B

Voluntary Seismic Upgrade Resources

This appendix presents leading documents used to guide seismic upgrade of structural configuration-related vulnerabilities in typical wood-frame houses. During post-earthquake damage investigations, it is possible that the structural consultant observes known seismic vulnerabilities (some of which were introduced in Chapter 2). Where a vulnerability is observed, but the damage repair by itself does not address the vulnerability, the structural consultant has the opportunity to recommend a voluntary seismic upgrade (retrofit) in the engineering report. Voluntary seismic upgrades are distinct from the code-triggered upgrades addressed in Appendix A. Vulnerabilities are most often related to the structural configurations of houses but can also be related to earthquake-induced permanent ground deformation. Voluntary seismic upgrades are usually outside of the scope of insurance coverage, so their cost is most often the responsibility of property owners; however, some insurers provide incentives for voluntary retrofits.

All of the following documents are published by FEMA and available online at no cost. Additional state and local documents addressing seismic upgrades can also be found online. Seismic upgrades to mitigate earthquake-induced permanent ground deformation should be discussed with a geotechnical consultant.

FEMA 530, *Earthquake Safety at Home* (FEMA, 2020). FEMA 530 is a general guideline to help manage earthquake risk to a house, including preparation and protection steps recommended to be taken before an earthquake. The document discusses common earthquake vulnerabilities in houses, with illustrations of potential damage and basic retrofit approaches.

FEMA P-1100, *Vulnerability-Based Seismic Assessment and Retrofit of One- and Two-Family Dwellings* (FEMA, 2018). This publication provides detailed assessment methods and retrofit designs for four common vulnerabilities: unbraced cripple walls around a crawlspace, living space over garage, hillside conditions, and brick chimneys and fireplace surrounds.

FEMA P-50, *Simplified Seismic Assessment of Detached, Single-Family, Wood-Frame Dwellings* (FEMA, 2012a). FEMA P-50 provides a checklist methodology that considers a house's construction, configuration, and location, resulting in a seismic performance grade and a prioritized list of seismic retrofit measures to improve the grade. The document is intended for use by anyone with a general knowledge of house construction, but it is most commonly used to gauge potential improvements, not as a retrofit design guide.

FEMA P-50-1, ***Seismic Retrofit Guidelines for Detached, Single-Family, Wood-Frame Dwellings*** **(FEMA, 2012b).** This publication provides background information for users of FEMA P-50, with more emphasis on retrofit considerations.

APPENDIX C

Earthquake Damage to Concealed Elements

This appendix provides brief summaries of collected sources of information regarding damage to concealed elements observed in laboratory testing. During an earthquake damage investigation, the structural consultant will most often observe finish materials and, in the absence of destructive investigation, will not directly observe the conditions of elements, such as structural sheathing and framing, concealed by the finish. Thus, the structural consultant will need to make judgements regarding the likelihood of there being damage to these concealed elements. In the Chapter 3 damage investigation checklists, the topic of damage to concealed elements is addressed by providing indicators of damage, where applicable. The following sources, among others, were considered in development of the Chapter 3 indicators.

Exterior Finish Materials

- Stucco (See also Appendix D)
  - **CUREE W-13 Specimens 17 and 18 (Gatto and Uang, 2002).** Testing of shear walls with stucco over plywood or OSB. Multiple studs were fractured near the base of studs. Damage should be visible at the base of the wall due to the stucco pulling away from the foundation. This might be interpreted as detachment of the stucco from the sheathing. This separation should trigger stucco removal and replacement. In doing so, the displacement of the sheathing should be noted, as it could indicate the need to open the sheathing and expose the framing damage below, generating further inspection openings.
  - **CUREE W-19 (Mosalam et al., 2002).** Shake table testing of weak-story building having stucco over plywood or OSB. Multiple studs were fractured similar to Gatto and Uang (2002).
- Wood Siding (Shiplap)
  - **PEER CEA (In progress).** Local splitting occurred in studs at sheathing nails. This was noted to greatly reduce the effort needed to withdraw nails. This resulted in loss of stiffness but is not believed to have significant loss of strength.
  - **PEER CEA (In progress).** Back face of siding became deformed around nail holes, with slots up to 3/4 inch created. These were primarily a concern in connection with greatly reduced stiffness, without appreciable loss of strength.
- Plywood Siding
  - **PEER CEA (In progress).** With improperly nailed siding, significant deformation of top and bottom plates, as well as window sills, occurred in line with vertical panel edges.

Interior Finish Materials

- Gypsum Wallboard (See also Appendix D)
  - **CUREE W-19 (Mosalam et al., 2002).** Gypsum nails cut slots up to 6 inches in length in the back side of gypboard, contributing to reduced stiffness. Little sign of damage was observed from the interior of the structure. Transient drift was the only indication that damage had occurred. This should be associated with a significant drop in stiffness. Renailing the gypboard between original nail locations should substantially restore stiffness and strength.
  - **PEER CEA (In progress).** Gypsum nails enlarged holes in back face around nails, cutting slots of up to 1 inch on the back side.
- Plaster over Wood Lath
  - **PEER CEA (In progress).** No deterioration to wood lath, nailing, or studs seen, even though the plaster fell completely off of the lath. No hidden damage of concern is anticipated to occur.

Exterior or interior finish materials other than those described in this appendix are generally anticipated to not have significant enough fastener size and spacing for the finish material to significantly damage the underlying structure. See Appendix D for further detail from laboratory testing.

APPENDIX D

Compiled Test Data: Seismic Strength and Stiffness

D.1 Introduction

This appendix summarizes available research illustrating relationships between earthquake damage patterns and the strength and stiffness of elements of the seismic-force-resisting system. Included is research in which cyclic testing of seismic-force-resisting elements was conducted and in which documentation was collected relating peak transient drift levels to the visual appearance of damage. This appendix uses this information in combination with hysteresis curves to establish relationships among: (a) visual appearance of damage; (b) peak transient drift; (c) an approximate location on the hysteresis curve; and (d) remaining strength relative to peak strength and reduction in stiffness.

From this information it can be estimated whether a given seismic-force-resisting element has been loaded up to or past peak capacity, in which case reduced capacity in a future earthquake might be anticipated unless the repair is able to restore strength. From this information it can also be estimated whether a given seismic-force-resisting element has been loaded to the extent that a significant decrease in stiffness has occurred, leading to the potential for significantly higher levels of drift in future earthquakes.

This appendix presents a limited number of sources representing best available information. Users of this information are cautioned that the actual performance of a given house may vary from the behavior illustrated in this appendix. Further, the research presented is limited by the need for the available documentation to include information pairing the visual appearance of damage observed during testing with associated peak transient drift levels.

The appendix focuses on the materials and systems most commonly occurring in the California housing inventory. In some instances, test data are for a wall with a combination of finish and sheathing materials.

D.2 Exterior Finish Materials or Exterior Wall Combined Materials

D.2.1 Stucco Exterior Combined with Gypsum Wallboard Interior

CUREE EDA-03, *Cyclic Behavior and Repair of Stucco and Gypsum Woodframe Walls: Phase* 1 (CUREE, 2003a) documents testing of single-story walls with stucco exterior finish and gypsum wallboard interior finish. The walls were tested cyclically using a displacement-based protocol. Dead load was used during testing, consistent with the walls being the first story of a two-story house. The tested walls included door and window openings. Detailing was provided with the intent of best replicating shear wall boundary conditions at the wall top, bottom, and ends. Information based on this testing is provided in Table D-1.

CUREE EDA-03 also evaluated the effectiveness of various repair methodologies to restore the strength and stiffness of wall specimens. Wall 3 and Wall 4 in the study were tested and repaired, with re-testing of repaired walls to four target drift ratios. Repairs to stucco and gypsum wallboard cracks were performed using the repair methodologies recommended in these *Engineering Guidelines*, including routing and filling cracks, resetting fasteners, installation of new fasteners, and other techniques. The repair methods were found to substantially restore the strength and stiffness of damaged walls. Figure D-1 shows the global wall response comparison at failure for Wall 1 and Wall 3d, with the latter specimen being a repaired wall specimen.

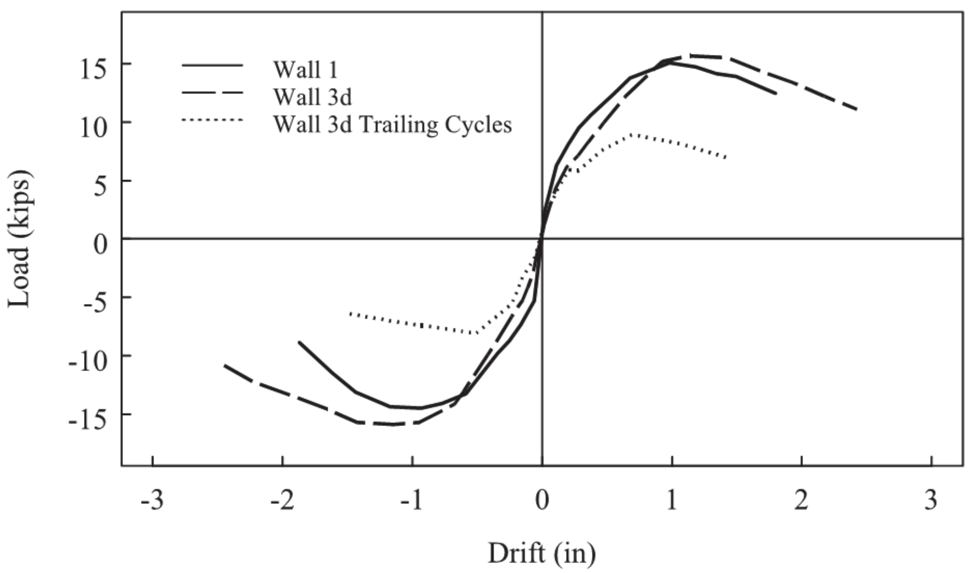


Figure D-1 CUREE EDA-03 Figure 6.46 showing global wall response comparison at failure for Wall 1 and Wall 3d, where Wall 3d was the repaired wall specimen.

CUREE EDA-07, *Cyclic Behavior and Repair of Stucco and Gypsum Sheathed Woodframe Walls: Phase II* (CUREE, 2003b), documents wall testing similar to the CUREE EDA-03 walls, except that superimposed load and wall boundary conditions were representative of the upper story of a two-story house. Information based on this testing is provided in Table D-2. The effectiveness of repair methodologies to restore the strength and stiffness of wall specimens was also evaluated in this study.

D.2.2 Shiplap Siding Exterior Combined with Plaster on Wood Lath Interior

Carroll (2006) documents testing of single-story walls with shiplap siding exterior finish and plaster on wood lath interior finish. The walls were tested cyclically using a displacement-based protocol. Wall samples were removed from existing residential structures set for demolition. Prior to the removal of the wall specimens, existing lath-and-plaster finishes were removed for asbestos abatement. New lath-and-plaster interior finishes were installed on removed wall samples. The new plaster finishes were reportedly constructed with "chicken wire" stapled to existing wood lath and an adhesive "bonder" applied to the wood lath. The construction of new plaster finishes was significantly different than the original plaster construction and the reader is cautioned that the test data may not be representative of typical construction. Walls were tested to failure before and after application of new lath-and-plaster interior finishes. Detailing to explicitly address wall boundary conditions was not incorporated in the specimens. Tested walls generally representing typical plaster on wood lath construction with similar test conditions were identified (Structure 1 and Structure 3 samples) and corresponding test results are reported in Table D-3.

D.3 Interior Finish Materials

D.3.1 Gypsum Wallboard Partition Walls

McMullin and Merrick (2002) documents testing of single-story gypsum wallboard sheathed partition walls. The walls were tested cyclically using a displacement-based protocol. The tested walls were sheathed on both sides with standard grade, 1/2-inch gypsum sheathing, and the wall specimens were designed to consider a range of test variables, including fastener type, fastener spacing, construction methods, wall boundary conditions, wall openings, and loading protocol. The testing program included identification of a series of damage thresholds and documentation of the corresponding peak drift ratio for each damage threshold during testing of each wall. Given that a range of test variables were evaluated during the testing program, tested walls representing typical gypsum wallboard construction with similar wall openings were identified (Walls 6, 14, 14R, 16, and 16R) and corresponding test results are reported in Table D-4. Notably, Wall 14R and Wall 16R are repaired wall test specimens.

D.4 Sheathing Materials

D.4.1 Wood Structural Panel Sheathing: Full-Height Walls

Gatto and Uang (2002) documents testing of single-story walls with wood structural panel sheathing. Test specimens included plywood sheathed walls using 15/32-inch Structural I Rated sheets and OSB using 3/8-inch-thick sheets. Edge nailing was typically doubled at the vertical exterior panel edges and was 2-inches on center, while panel edge nailing elsewhere was 4-inches on center. Detailing to explicitly address wall boundary conditions was not incorporated in the test specimens, so walls had little perimeter confinement. The walls were tested cyclically using a displacement-based protocol. Tested wall assemblies included walls without exterior or interior finishes, walls with interior gypsum wallboard, and walls with exterior stucco applied over the sheathed wall side.

Gatto and Uang (2002) noted that test specimens with sheathing or finishes on only one side exhibited stud twisting during testing and that behavior was believed to be primarily a result of eccentricity imposed on the framing. For stucco wall specimens, fracturing of wall studs was also observed during testing.

Due to the lack of confinement at wall boundaries, it is believed that the peak capacities for the stucco walls in this test series were achieved at higher drift ratios than expected. Information based on this testing is provided in Table D-5 through Table D-9.

Pardoen et al. (2003) documents testing of single-story walls with wood structural panel sheathing. The walls were tested cyclically using a displacement-based protocol. The tested walls included “pedestrian” door and garage door openings. Test specimens were constructed of OSB sheathing, with and without interior gypsum wallboard. Information based on this testing is provided in Table D-8 and Table D-9.

Pardoen et al. (2003) also tested two-story wall specimens and concluded that the addition of the second story did not significantly affect the first-story response. The first-story response for the two-story wall specimens was very similar to the response of similar one-story wall specimens.

D.4.2 Diagonal Lumber Sheathed Exterior: Full-Height Walls

Ni and Karacabeyli (2007) documents testing of single-story diagonal lumber sheathed walls. The walls were tested cyclically using a displacement-based protocol. Tested walls included specimens with a single layer of lumber sheathing and specimens with a double layer of lumber sheathing on the sheathed wall side. The testing program investigated the effects of wall boundary conditions on in-plane shear performance, with tested conditions including the detailing of tie-down connections at wall ends, detailing of bottom (sill) plate of walls, and application of vertical wall load. Information based on testing of single-layer diagonal lumber sheathed walls is provided in Table D-10, and information based on testing of double-layer diagonal lumber sheathed walls is provided in Table D-11.

D.4.3 Horizontal Lumber Sheathed Exterior: Full-Height Wall

Ni and Karacabeyli (2007) documents testing of a single-story horizontal lumber sheathed wall with interior gypsum wallboard. The wall was tested cyclically using a displacement-based protocol. Information based on this testing is provided in Table D-12.

D.5 Connectors

D.5.1 Wall Sill Plate Anchorage

Mahaney and Kehoe (2002) documents testing of wall sill plate-to-foundation anchorage. Test specimens consisted of walls connected to a simulated concrete footing that were tested cyclically using a force-controlled loading protocol. Walls were sheathed on one face with plywood sheathing. The specimens were designed to consider a range of test variables, including dead load, anchor bolt washer size, sill plate width and thickness, anchor bolt size, and sill plate wood species. Additionally, the test program evaluated the effect of shear wall overturning moments on the performance of the wall sill plate anchorage by varying the height at which the horizontal load was applied during testing. Two test setups were developed: Test Setup 1 had the horizontal load applied 1 foot above the base of the wall, and Test Setup 2 had the horizontal load applied 8 feet above the base of the wall.

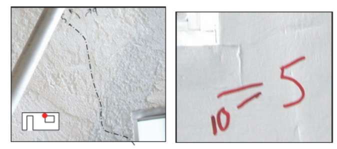
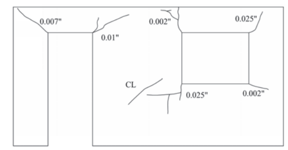
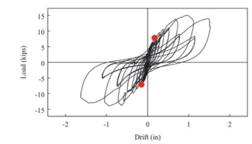
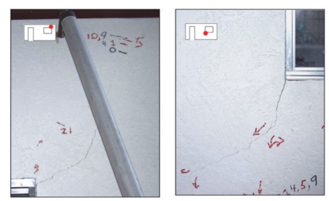
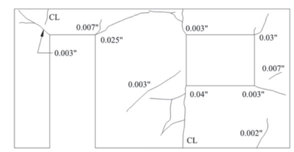
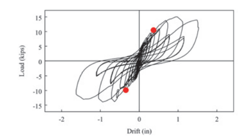
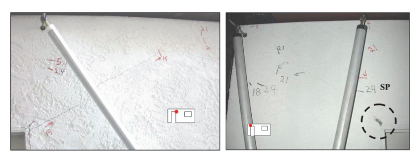
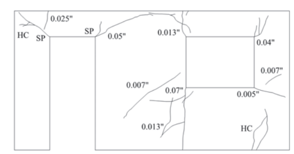
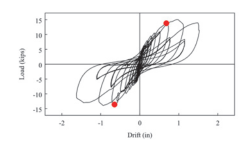
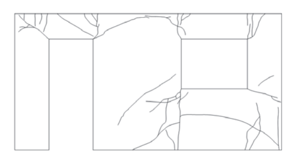
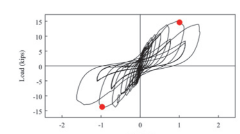
Test specimens were organized into representative groups based on the detailing of the wall specimen and test setup used. Group 1 represented testing of wall specimens anchored to concrete with steel threaded rod anchor bolts and using Test Setup 1, where the load was applied 1 foot above the wall base. Information based on this test group for wall specimens with 2× sill plates is provided in Table D-13. Group 3 represented testing of wall specimens anchored to concrete with steel threaded rod anchor bolts and using Test Setup 2, where the load was applied 8 feet above the wall base. Information based on this test group is provided in Table D-14.

D.5.2 Diaphragm Connections

Ficcadenti et al. (2004) documents testing of shear transfer connections between diaphragms and shear walls. The tested diaphragm and shear wall assemblies were designed to be stronger than the tested shear transfer connections for the assemblies. The test program evaluated three different shear transfer connection types: 2×12 framing with toe-nailed connections, 2×12 framing with proprietary framing clips, and engineered I-joist framing with flange-nailed connections. Test specimens were tested cyclically using a displacement-based loading protocol. While the testing program was primarily focused on unblocked diaphragms, three specimens with blocked diaphragms were included in the test program.

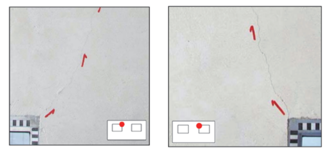
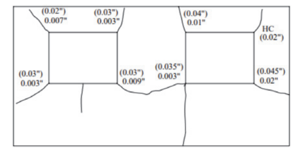
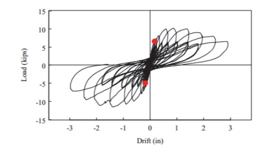
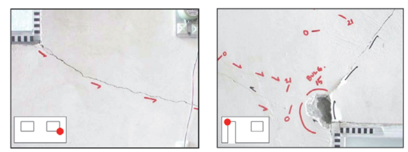
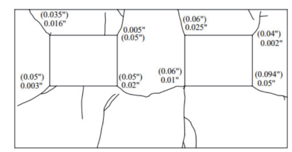
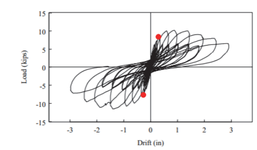
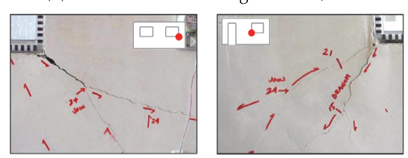
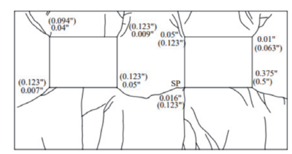
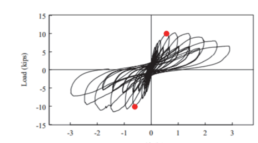
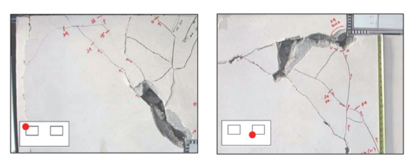
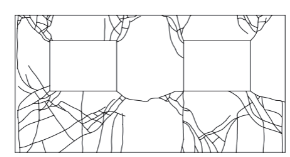
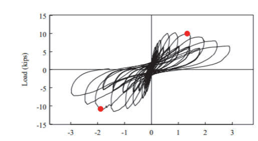
Information based on testing of toe-nailed shear transfer connections for unblocked diaphragms is provided in Table D-15, and information based on testing of proprietary framing clips for unblocked diaphragms is provided Table D-16.

Table D-1 Earthquake Damage Relationships: Exterior Stucco and Interior Gypsum Wallboard, Part 1

Wall Material	*Damage Description*	*Damage Appearance: Local Appearance*	*Damage Appearance: Overall Pattern (crack widths shown were measured when the wall was returned to zero force)*	*Peak Transient Drift When Observed (% of story height)*	*Hysteresis Curve*
Exterior stucco and interior gypsum wallboard	Wall behavior up to 0.2% drift was characterized by a very stiff nearly linear-elastic response	 Wall 2 cracking up to 0.2% drift	 Wall 2	0.2%	
	Wall behavior from 0.2% to 0.4% drift was characterized by some softening of the wall stiffness, extension of cracks in length and width, development of new cracks branching off of existing cracks, and no deterioration of wall response during trailing cycles	 Wall 2 stucco cracking up to 0.4% drift	 Wall 2	0.4%	
	Wall behavior from 0.4% to 0.7% drift was characterized by softening of the wall stiffness, extension of cracks in length and width, development of new cracks, and very slight deterioration of wall response in trailing cycles	 Wall 2 stucco cracking up to 0.7% drift	 Wall 2	0.7%	
	Wall behavior from 0.7% drift up to ultimate strength was characterized by softening of the wall stiffness, extension of cracks in length and width, development of new cracks, and more sever deterioration of behavior during trailing cycles	 Wall 1 stucco cracking at failure	 Wall 2 stucco cracking at failure (no crack widths provided)	ultimate	

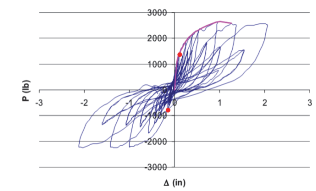
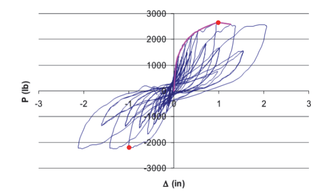
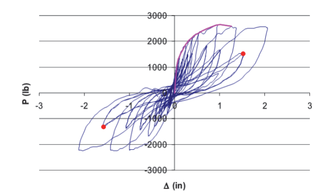
Note: Information based on tests of Wall 1 and Wall 2 from CUREE EDA-03.

Table D-2 Earthquake Damage Relationships: Exterior Stucco and Interior Gypsum Wallboard, Part 2

Wall Material	*Damage Description*	*Damage Appearance* — *Local Appearance*	*Damage Appearance* — *Overall Pattern* (*maximum and residual stucco crack widths were measured at all drift levels and reported*)	*Peak Transient Drift When Observed* (*% of story height*)	*Hysteresis Curve*
Exterior stucco and interior gypsum wallboard	Wall behavior up to 0.2% drift was characterized by a very stiff nearly linear-elastic response	 (a) (b) Wall 5 stucco cracking: (a) north window, (b) south window	 Wall 5	0.2%	
	Wall behavior from 0.2% to 0.4% drift is characterized by some softening of the wall stiffness, extension of cracks in length and width, development of new cracks branching off primary cracks, and no deterioration of wall response during trailing cycles	 (a) (b) (a) Wall 5 stucco cracking after 0.4% drift, (b) Wall 6 stucco cracking after 0.6% drift	 Wall 5	0.4%	
	Wall behavior from 0.4% to 0.7% drift is characterized by softening of the wall stiffness, extension of cracks in length and width, development of new cracks, and very slight deterioration of wall response during trailing cycles	 (a) (b) (a) Wall 5 stucco cracking for 0.7% drift, (b) Wall 6 stucco damage for 0.9% drift	 Wall 5	0.7%	
	Wall behavior from 0.7% drift up to ultimate strength is characterized by softening of the wall stiffness, extension of cracks in length and width, development of new cracks, and significant deterioration of behavior during trailing cycles	 Wall 5 stucco damage at failure	 Wall 5 stucco damage at failure (no crack widths provided)	ultimate	

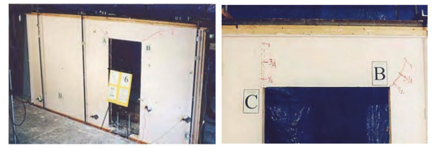
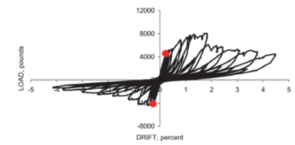
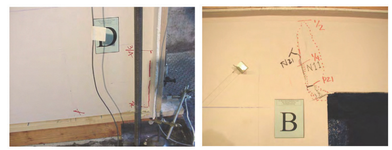
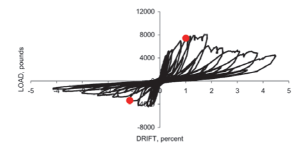
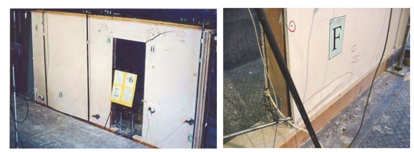
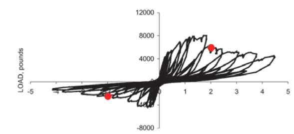
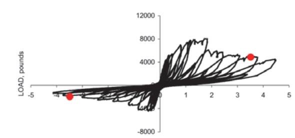
Note: Information based on tests of Walls 5, 6, 7, and 8 from CUREE EDA-07.

Table D-3 Earthquake Damage Relationships: Exterior Ship Lap Siding and Interior Plaster of Wood Lath

Wall Material	*Damage Description*	*Damage Appearance*	*Peak Transient Drift When Observed (% of story height)*	*Hysteresis Curve*
Exterior ship lap siding and interior plaster on wood lath	Wall behavior up to 0.1% drift was characterized by a very stiff nearly linear-elastic response	Not Provided	0.1%	
	Wall behavior at ultimate was characterized by softening of the wall stiffness, peak in-plane shear capacities ranging from 2.5 kips to 3 kips, and peak drift ranging from 0.5% to 1%	Not Provided	0.5% to 1%	
	Wall behavior at failure was characterized by separation of the plaster from the double top plate of the shear wall followed by the double top plate shearing off of the top of the wall	 Typical wall failure resulted in the double top plate shearing off of the top of the wall	> 0.75%	

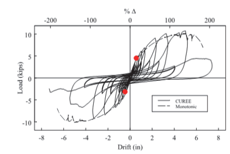
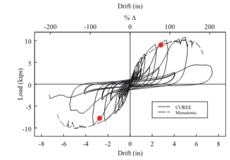
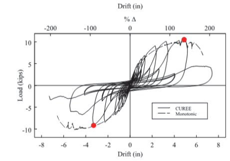
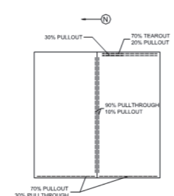
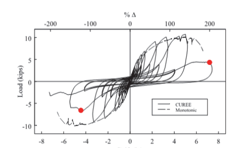
Note: Information based on tests of Structure 1 and Structure 3 plaster on wood lath walls from Carroll (2006)

Table D-4 Earthquake Damage Relationships: Gypsum Wallboard

Wall Material	Damage Description	Damage Appearance (photographs are inclusive of all walls tested by McMullin and Merrick)	Peak Transient Drift Range When Observed (% of story height)	Hysteresis Curve
Gypsum wallboard	Wall behavior up to 0.25% drift was characterized by a very stiff nearly linear-elastic response. Cracking at the door openings usually initiated at drift ranging from 0.04% to 0.24%	 (a) (b) (a) Damage of Test 6 at 0.75%, (b) Cracking at window at location of vertical butt joint	0.04% to 0.24%	
	Wall behavior up to 1.0% drift was characterized by some softening of the wall stiffness and extension of cracks in length and width. Paint cracking at fastener heads, local buckling of panels at wall penetrations and cracking of joints at out-of-plane walls usually initiated at drift ranging from 0.2% to 0.9%	 (a) (b) (a) Cracking at vert. joint to int. wall (Wall 1), (b) Local buckling of panel at door (Wall 15R)	0.2% to 0.9%	
	Wall behavior up to 2% drift is characterized by softening of the wall stiffness and some deterioration of wall response in trailing cycles. Crushing of wallboard at the perimeter usually initiated at drift ranging from 0.5% to 2%	 (a) (b) (a) Damage of Test 6 at 2%, (b) crushing and buckling of wallboard at door jamb (Wall 4)	0.5% to 2%	
	Wall behavior at drifts greater than 2.0% was characterized by continued softening of the wall stiffness and deterioration of wall response in trailing cycles. Global buckling of wall panels usually initiated at drift ranging from 1.9% to 3.5%	 (a) (b) (a) Buckling of large region of panel (Wall 16R), (b) loss of entire panel (Wall 17)	1.9% to 3.5%	

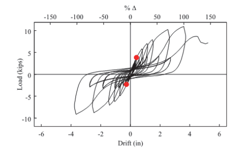
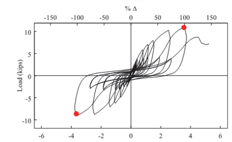
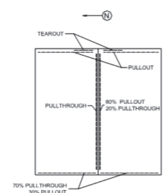
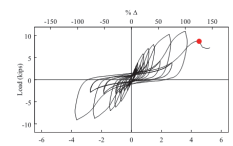
Note: Information based on tests of Walls 6, 14, 14R, 16, and 16R from McMullin and Merrick (2002).

Table D-5 Earthquake Damage Relationships: Wood Structural Panel (Plywood and OSB)

Wall Material	*Damage Description*	*Damage Appearance*		*Peak Transient Drift When Observed (% of story height)*	*Hysteresis Curve*
		Local Appearance	*Overall Pattern*		
Wood structural panel (plywood and OSB)	Wall behavior up to 0.5% drift was characterized by a very stiff nearly linear-elastic response	Not Provided	Not Provided	0.5%	
	Onset of observable nail damage for Test 6 started at a displacement of 70% of the reference displacement (roughly 2.5% story drift)	Not Provided	Not Provided	2.5%	
	Wall behavior at ultimate was characterized by softening of the wall stiffness, peak in-plane shear capacities ranging from 9 kips to 11 kips, and peak drift ranging from 5% to 6.5%	Not Provided	Not Provided	5% to 6.5%	
	Wall behavior at failure included: sheathing separation, sheathing buckling, sheathing rotation, stud separation from sill, tie-down twisting	 Damage photographs for Test 6: plywood with CUREE protocol showing sheathing buckling		> 6.5%	

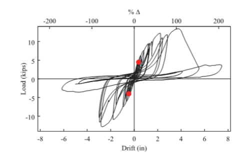
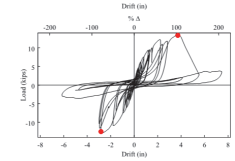
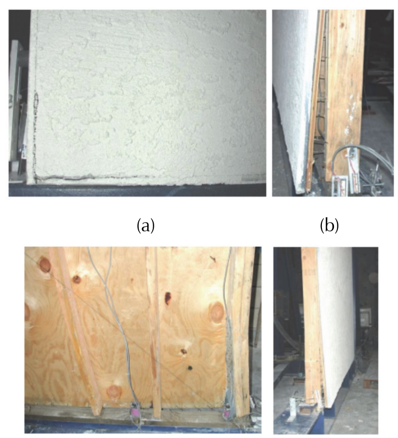
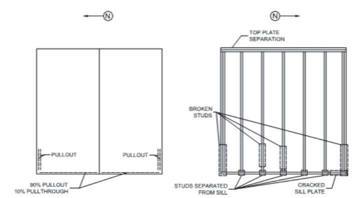
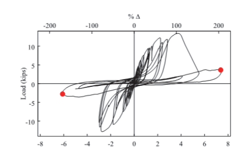
Note: Information based on Test 2 and Test 6 from Gatto and Uang (2002).

Table D-6 Earthquake Damage Relationships: Wood Structural Panel (Plywood and OSB) and Interior Gypsum Wallboard

Wall Material	Damage Description	Damage Appearance: Local Appearance	Damage Appearance: Overall Pattern	Peak Transient Drift When Observed (% of story height)	Hysteresis Curve
Wood structural panel (plywood and OSB) and interior gypsum wallboard	Wall behavior up to 0.5% drift was characterized by a very stiff nearly linear-elastic response	Not Provided	Not Provided	0.5%	
	Wall behavior at ultimate was characterized by softening of the wall stiffness, peak in-plane shear capacities ranging from 10 kips to 11 kips, and peak drift ranging from 3% to 5%	Not Provided	Not Provided	3% to 5%	
	Wall behavior at failure included: top plate damage, top plate separation, split sill plate, sheathing separation, sheathing buckling, and nail tearout	 (a) (b)  (a) (b) Damage photographs for Tests 13 and 14: (a) top plate separation, (b) split sill plate, (c) horizontally split sill plate, and (d) out-of-plane damage		> 5%	

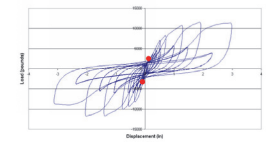
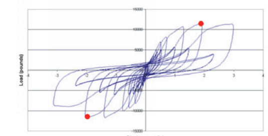
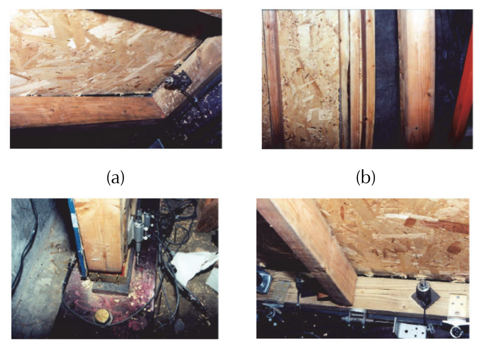
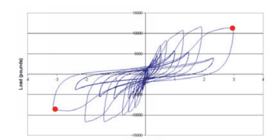
Note: Information based on Test 13 and Test 14 from Gatto and Uang (2002).

Table D-7 Earthquake Damage Relationships: Exterior Stucco over Wood Structural Panel (Plywood)

Wall Material	*Damage Description*	*Damage Appearance*		*Peak Transient Drift When Observed (% of story height)*	*Hysteresis Curve*
		Local Appearance	*Overall Pattern*		
Exterior stucco over wood structural panel (plywood)	Wall behavior up to 0.5% drift was characterized by a very stiff nearly linear-elastic response	Not Provided	Not Provided	0.5%	
	Wall behavior at ultimate was characterized by softening of the wall stiffness, peak in-plane shear capacities ranging from 11 kips to 12 kips, and a peak drift around 4%	Not Provided	Not Provided	4%	
	Wall behavior at failure included: stucco damage, sheathing separation, tie-down twisting	 (a) (b) (c) (d) Damage photographs for Test 18: plywood + stucco with CUREE dynamic protocol—(a) stucco damage, (b) sheathing separation, (c) tie-down twisting, and (d) sheathing separation	 Test 18 West Wall	> 4%	

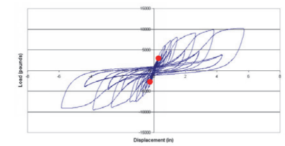
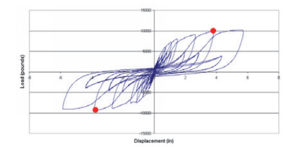
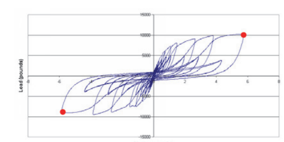
Note: Information based on Test 18 from Gatto and Uang (2002).

Table D-8 Earthquake Damage Relationships: Wood Structural Panel (OSB)

Wall Material	*Damage Description*	*Damage Appearance*	*Peak Transient Drift When Observed (% of story height)*	*Hysteresis Curve*
Wood structural panel (OSB)	Wall behavior up to 0.1% drift was characterized by a very stiff nearly linear-elastic response	Not Provided	0.1%	
	Wall behavior at ultimate was characterized by softening of the wall stiffness, peak in-plane shear capacities ranging from 10 kips to 12 kips, and peak drift ranging from 2% to 3%	Not Provided	2% to 3%	
	Wall behavior at failure included: damage to edge nailing, sheathing separation from framing, sheathing separation at joints, splitting of framing, bowing of framing, and separation of framing members	 (a) (b) (c) (d) Group 8A Specimen: (a) OSB pulled away from frame, (b) exterior post stud bowed, (c) sill plate split, and (d) separation of OSB from sill	> 3%	

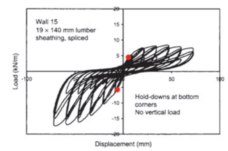
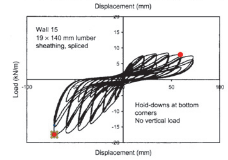
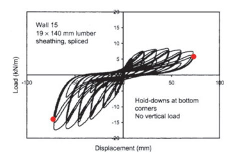
Note: Information based on tests of Groups 4, 6, 8, and 10 from Pardoen et al. (2003).

Table D-9 Earthquake Damage Relationships: Wood Structural Panel (OSB) and Interior Gypsum Wallboard

Wall Material	*Damage Description*	*Damage Appearance*	*Peak Transient Drift When Observed (% of story height)*	*Hysteresis Curve*
Wood structural panel (OSB) and interior gypsum wallboard (GWB)	Wall behavior up to 0.2% drift was characterized by a very stiff nearly linear-elastic response	Not Provided	0.2%	
	Wall behavior at ultimate was characterized by softening of the wall stiffness, peak in-plane shear capacities ranging from 9 kips to 10 kips, and peak drift ranging from 3% to 4%	Not Provided	3% to 4%	
	Wall behavior at failure included: damage to edge nailing, splitting of framing, tearing in gypsum wallboard at openings, tearing and cracks at OSB at openings	 (a) (b) Groups 22 Specimens: (a) sill plate split and (b) crack in GWB at opening (scaffolding in foreground)  (c) (d) Group 23 Specimens: (c) tear in OSB at opening and (d) complete separation of GWB from frame (scaffolding in foreground)	> 4%	

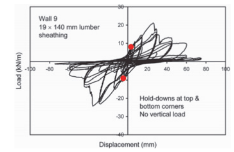
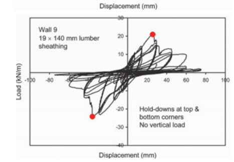
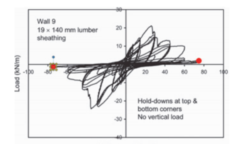
Note: Information based on tests of Group 22 and Group 23 from Pardoen et al. (2003).

Table D-10 Earthquake Damage Relationships: Single-Layer Diagonal Lumber Sheathing

Wall Material	*Damage Description*	*Damage Appearance*	*Peak Transient Drift When Observed (% of story height)*	*Hysteresis Curve*
Single-layer diagonal lumber sheathing	Wall behavior up to 0.2% drift was characterized by a very stiff nearly linear-elastic response	Not Provided	0.2%	
	Wall behavior at ultimate was characterized by softening of the wall stiffness, peak in-plane shear capacities ranging from 500 plf to 1,200 plf, and peak drift ranging from 2% to 3%	Not Provided	2% to 3%	
	Wall behavior at failure was generally characterized by two types of failure modes: failure of perimeter nails or failure of walls at corners before perimeter nail capacity could be developed	 Failure modes of tested shear walls	> 3%	

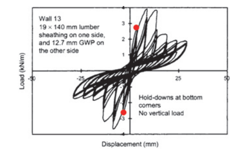
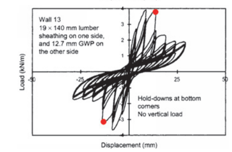
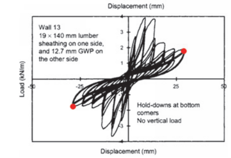
Note: Information based on tests of Walls 4, 5, 6, 10, 11, 15, and 16 from Ni and Karacabeyli (2007).

Table D-11 Earthquake Damage Relationships: Double-Layer Diagonal Lumber Sheathing

Wall Material	*Damage Description*	*Damage Appearance*	*Peak Transient Drift When Observed (% of story height)*	*Hysteresis Curve*
Double-layer diagonal lumber sheathing (diagonal sheathing oriented 90 degrees to each other)	Wall behavior up to 0.2% drift was characterized by a very stiff nearly linear-elastic response	Not Provided	0.2%	
	Wall behavior at ultimate was characterized by some softening of the wall stiffness, peak in-plane shear capacities ranging from 1,400 plf to 2,200 plf, and peak drift ranging from 1% to 2%	Not Provided	1% to 2%	
	Wall behavior at failure was generally characterized by splitting failure at bottom (sill) plates	 Splitting failure at double bottom plate for tested shear wall	> 2%	

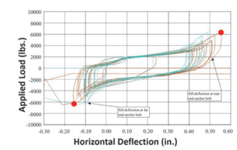
Note: Information based on tests of Walls 7, 8, and 9 from Ni and Karacabeyli (2007).

Table D-12 Earthquake Damage Relationships: Single-Layer Straight Lumber Sheathing and Interior Gypsum Wallboard

Wall Material	*Damage Description*	*Damage Appearance*	*Peak Transient Drift When Observed (% of story height)*	*Hysteresis Curve*
Single-layer straight lumber sheathing and interior gypsum wallboard	Wall behavior up to 0.1% drift was characterized by a very stiff nearly linear-elastic response	Not Provided	0.1%	
	Wall behavior at ultimate was characterized by some softening of the wall stiffness, a peak in-plane shear capacity of roughly 250 plf, and a peak drift of about 0.6%	Not Provided	0.6%	
	Story drift at failure was roughly 1%	Not Provided	> 0.6%	

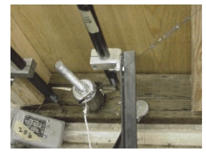
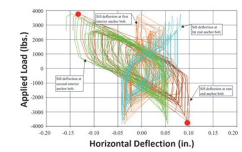
Note: Information based on tests of Wall 13 from Ni and Karacabeyli (2007).

Table D-13 Earthquake Damage Relationships: Sill Plate-to-Foundation Anchorage, Part 1

Specimen	*Damage Description*	*Damage Appearance*	*Sill Horizontal Displacement at Anchor Bolt (test group average)*	*Hysteresis Curve (load vs. sill horiz. disp. at anchor bolt)*
Sill plate-to-foundation anchorage with steel threaded rod anchor bolts and 2x sill plates (horizontal loading applied 1 foot above base of wall)	Sill plate-to-foundation anchorage behavior for loads approaching ultimate capacity was typically characterized by yielding of the anchor bolts in bending and elongation of the bolt holes at the sill plate	Not Provided	Not Provided	Not Provided
	Sill plate-to-foundation anchorage behavior at failure was typically characterized by splitting of the sill plate. This occurred as net uplift forces on the test specimens increased and the sill plate bent and twisted due to uplift forces in the plywood and the inherent eccentricity between the plywood and the anchor bolts	 (a) Test 1 Failure  (b) Test 2 Failure  (c) Test 6 Failure  (d) Test 10 Failure	0.25 inches to 0.50 inches	 Test 10

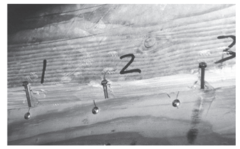
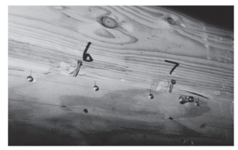
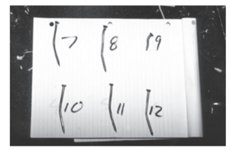
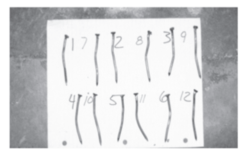
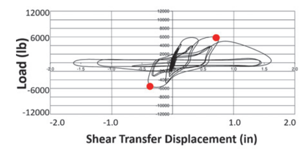
Note: Information based on tests of Group 1 through Group 11 from Mahaney and Kehoe (2002).

Table D-14 Earthquake Damage Relationships: Sill Plate-to-Foundation Anchorage, Part 2

Specimen	*Damage Description*	*Damage Appearance*	*Sill Plate Horizontal Displacement at Anchor Bolt (test group average)*	*Hysteresis Curve (load vs. sill horiz. disp. at anchor bolt)*
Sill plate-to-foundation anchorage with steel threaded rod anchor bolts and no wall tie-downs (horizontal loading applied 8 feet above base of wall)	Sill plate-to-foundation anchorage behavior at failure was typically characterized by splitting of the sill plate. This occurred as net uplift forces on the test specimens increased and the sill plate bent and twisted due to uplift forces in the plywood and the inherent eccentricity between the plywood and the anchor bolts	 (a) Test 21 failure  (b) Test 28 failure  (c) Test 29 failure  (d) Test 38 failure	0.10 inches	 Test 21C

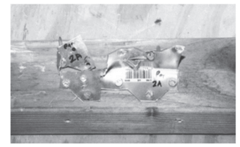
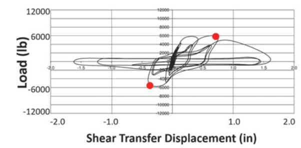
Note: Information based on Group 3, Tests 20, 21, 24–36, 38, and 40, from Mahaney and Kehoe (2002).

Table D-15 Earthquake Damage Relationships: Toe-Nailed Connections for Unblocked Diaphragms to Shear Walls

Specimen	*Damage Description*	*Damage Appearance*	*Shear Transfer Connection Displacement at Ultimate*	*Hysteresis Curve (load vs. connection displacement)*
Toe-nailed connections for unblocked diaphragms to shear walls	Shear transfer connection behavior at failure was typically characterized by a combination of joist or block splitting and nails undergoing bending. Tested in-plane shear capacities ranged between 500 plf to 700 plf and shear connection displacements at ultimate ranged from 0.25 inches to 1 inch.	  Specimen TN1-A   Specimen TN1-B   Spciemen TN2-A	0.25 inches to 1 inch	 Specimen TN1-A Toe-nailed shear transfer connection to parallel joists with 16-penny nails at 8-inches on center

Note: Information based on tests of Specimens TN1-A, TN2-A, and TN2-B from Ficcadenti et al. (2004).

Table D-16 Earthquake Damage Relationships: Proprietary Framing Clip Connections for Unblocked Diaphragms to Shear Walls

Specimen	*Damage Description*	*Damage Appearance*	*Shear Transfer Connection Displacement at Ultimate*	*Hysteresis Curve (load vs. connection displacement)*
Proprietary framing clip connections for unblocked diaphragms to shear walls	Shear transfer connection behavior at failure was typically characterized by buckling of the framing clip followed by the withdrawal of nails at the connection. Tested in-plane shear capacities ranged between 750 plf to 850 plf and shear connection displacements at ultimate ranged from 0.3 inches to 0.75 inches	  Specimen FC1-A   Specimen FC1-B (left), Specimen FC2-A (right)   Specimen FC2-B	0.3 inches to 0.75 inches	 Specimen FC1-A Framing clip shear transfer connection to parallel joists with three "A35" clips

Note: Information based on tests of Specimens FC1-A, FC1-B, FC2-A, and FC2-B from Ficcadenti et al. (2004).

APPENDIX E

Surface Fault Rupture

Appendix E presents a short description of analysis that can be performed to evaluate if observed ground deformations are potentially related to distributed surface fault rupture.

Distributed surface fault rupture has been the subject of considerable research for the following conditions: (1) distributed rupture involving discrete breaks across discontinuities for normal fault earthquakes; and (2) distributed rupture involving ground warping for large-magnitude strike-slip earthquakes. Data and models for distributed surface rupture for other conditions are under presented by Petersen et al. (2004).

The data for distributed rupture associated with normal fault earthquakes was compiled as part of studies for the Yucca Mountain project in Nevada. Figure E-1 shows the probability of distributed rupture as a function of magnitude, distance from the primary rupture, and location of the site on the hanging or foot walls (Youngs et al., 2003). The probabilities shown in the figure are for the occurrence of distributed rupture across a discontinuity. Accordingly, geologic discontinuities would need to be present at a particular site under investigation in order for the data in Figure E-1 to be appropriately applied, and the probabilities obtained only apply to displacements across the discontinuity. From Figure E-1, the probability of surface rupture is seen to be much higher on the hanging wall than on the foot wall, to increase with magnitude, and to decrease with distance. The probability of distributed surface rupture becomes very small for distances greater than about 10 miles (15 km). The orientation of the discontinuity strike with respect to the strike of the primary fault also affects the probability of distributed rupture on the discontinuity. As shown in Figure E-2, the probability of distributed rupture decreases as the difference between strikes increases. Importantly, the data in Figure E-1 and Figure E-2 are only applicable to normal fault earthquakes.

The data for distributed rupture associated with strike-slip earthquakes is more limited than that for normal fault earthquakes. Geomatrix Consultants investigated the warping of fences crossing the primary rupture of the San Andreas fault from the 1906 San Francisco earthquake (Figure E-3). The results show the variation of ground warping with distance from the primary trace, where the ground distortion is the amount of right-lateral slip observed per length of fence segment and the length of each line indicates the length of fence segment for which deformation was measured. Within Figure E-3, the horizonal lines at 1% and between 4% and 5% refer to distortion amounts considered by the investigators to be insignificant or indicative of a distinct offset across a discontinuity, respectively. These data may not be appropriate for other faults or other magnitude earthquakes. The data apply for distortion adjacent to relatively straight-ruptured fault segments and are not applicable in step-over zones.

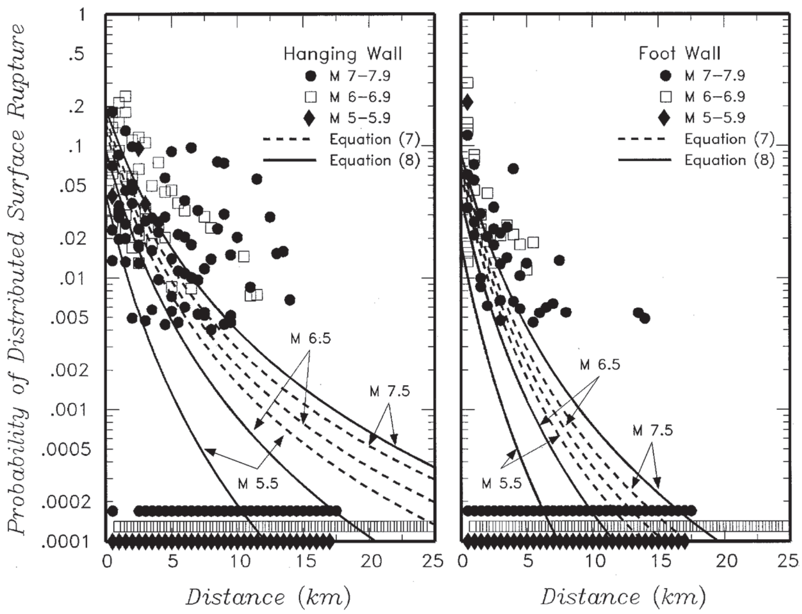


Figure E-1 Probability of slip for distributed faulting across a discontinuity conditioned on magnitude (*M*), distance from primary rupture, and location of site on hanging or foot wall (Youngs et al., 2003).

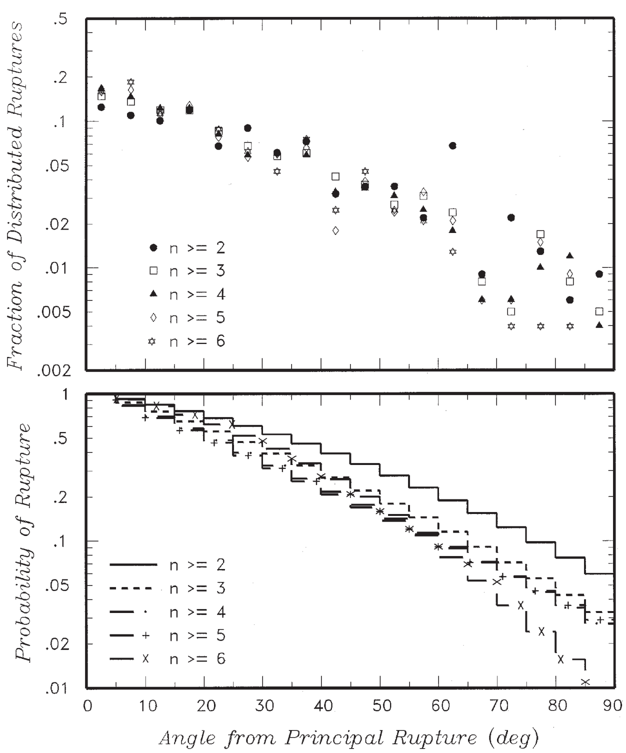


Figure E-2 Effect of angle on probability of rupture: (top) fraction of distributed ruptures versus the angle between the strike of the distributed fault and the strike of the principal rupture; and (bottom) probability of rupture versus the angle between the strike of the distributed fault and the strike of the principal rupture, normalized to 1 at an angle of 0 degrees (Youngs et al., 2003).

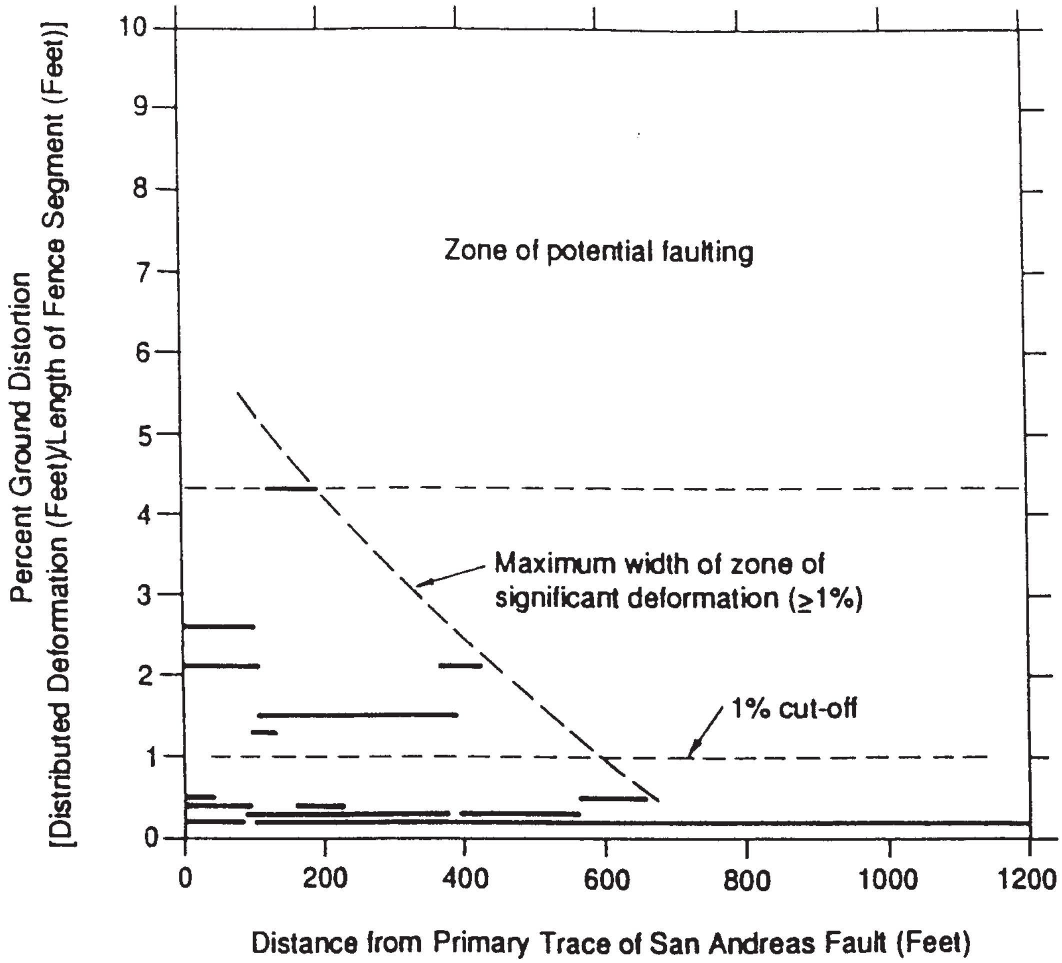


Figure E-3 Percent of ground distortion as function of distance from primary trace of the 1906 San Francisco, California earthquake rupture on the San Andreas fault (figure credit: D. Wells, *personal communication*, 2002).

APPENDIX F

Liquefaction

Appendix F presents descriptions of screening and detailed liquefaction analyses.

F.1 Screening Analysis for Liquefaction

The purpose of a screening analysis is to determine if a site has obvious characteristics that would indicate that liquefaction could not have occurred. As discussed in Chapter 5, requisite conditions for the occurrence of liquefaction are that the soil material is susceptible to liquefaction, and earthquake loading is sufficiently large to trigger liquefaction. Soil materials considered susceptible to liquefaction are generally sands, gravels, and low plasticity silts located below the ground water table at the time of the earthquake. Such conditions are generally found in geologically young alluvial deposits or artificial fills in areas with shallow groundwater. Aspects of the earthquake shaking that influence the potential that liquefaction will be triggered for a given soil condition include the amplitude and duration. The above considerations may be utilized to perform an approximate screening analysis of whether liquefaction was possible at a site.

The technical literature contains numerous guidelines for screening liquefaction hazards. These guidelines generally consider the *potential* for liquefaction to occur for some reasonable set of assumptions about future earthquake loading. As a result, these guidelines consider a different scenario than what is considered in this document, which assumes that an earthquake has already occurred and that earthquake parameters are well defined and available.

Specific guidelines for assessing liquefaction susceptibility are presented by CDMG (1997) and Martin and Lew (1999) and are adapted here to provide general guidelines for post-earthquake site screening.

If the estimated ground water level at the time of the earthquake is determined to be deeper than 50 feet below the existing ground surface, then further liquefaction assessments are not required. Ground water levels may be determined from published data or site-specific exploration.

If “bedrock” or similar lithified formational material is present at the surface of the site, those materials need not be considered liquefiable and no analysis of their liquefaction potential is necessary. A list of those local formations that (for purposes of a preliminary screening) are considered to be “bedrock” may be available from the local building official or the state geological survey.

If high fines content soil materials (> 35%) are present at the site exploration, it is possible that those materials are not susceptible to liquefaction. Bray and Sancio (2006) provide criteria for assessing the liquefaction susceptibility of fine-grained soils based on liquid limit (*LL*), plasticity index (*PI*), and water content (w_c), as summarized below; Boulanger and Idriss (2006) provide criteria based on *PI*. Engineering judgement should always be exercised in use of these assessment methods and the technical consultant is referred to the original publications for details on each method.

- Bray and Sancio (2006): Not Susceptible (w_c/LL < 0.8 and PI > 18), Moderately Susceptible (w_c/LL > 0.8 and 12 < PI < 18), and Susceptible (w_c/LL > 0.85 and PI < 12), transitional response across boundaries rather than distinct change at the exact boundary
- Boulanger and Idriss (2006): Soil with PI > 7 exhibits "clay-like" behavior, intermediate response in a transitional zone for soil with lower *PI*

Soils that are assessed to be "not susceptible" or likely to exhibit "clay-like" response are unlikely to liquefy in the classic sense. However, the absence of liquefaction susceptibility on the basis of these criteria does not necessarily imply that permanent ground deformation cannot occur—cyclic softening of saturated clayey soil can still lead to permanent ground deformation, especially in the presence of a driving static shear stress, such as from a foundation or slope (see Section F.4 for details on shear strength parameter selection for stability analyses in these materials). An early method for assessing liquefaction susceptibility of fine-grained soil was the so-called Chinese criteria originally presented by Seed and Idriss (1982) and subsequently re-stated in references, such as Martin and Lew (1999) and Youd et al. (2001). However, the Chinese criteria were found to be ineffective at distinguishing liquefiable and non-liquefiable soils by Sancio et al. (2002) and Stewart et al. (2003) and should no longer be used.

If the corrected standard penetration blow count, $(N_1)_{60}$, is greater than or equal to 30 in all samples, and a sufficient number and spacing of samples is available, further liquefaction assessments are not required.

If the screening investigation clearly indicates the absence of liquefaction susceptibility, further analysis is not required of the site for liquefaction.

F.2 Subsurface Exploration and Laboratory Testing

The objective of subsurface exploration and laboratory testing for liquefaction studies is to characterize the soil stratigraphy, evaluate the depth to groundwater, evaluate soil index properties, and develop penetration resistance measures for the susceptible soil layers. For sites without significant gravel deposits, subsurface exploration will most typically involve the drilling of boreholes, possibly supplemented by the advancing of cone penetration test (CPT) soundings. For gravelly soil sites, Becker Penetration Tests (BPT) may be performed in lieu of borings with standard penetration test (SPT) or CPT. In situ measurements of soil shear wave velocity (V_s) can be used to supplement, but generally not replace, traditional penetration resistance testing. Advantages and limitations of each test are summarized in Table F-1. The penetration resistance tests most commonly used in practice are SPT and CPT.

The CPT offers several advantages relative to SPT that are related to the test method's speed, relatively low cost, and its ability to provide a nearly continuous penetration resistance profile. However, CPT soundings typically do not provide samples. Accordingly, it is preferable to perform both CPT and borings with SPT. Martin and Lew (1999) recommends that as a minimum, one soil boring be performed next to a CPT sounding (using engineering judgment to determine an appropriate distance between the boring and sounding) to confirm soil types and verify liquefaction resistance interpretations based on CPT data. Additional soil borings may be necessary depending on the size of the site and variation of subsurface conditions.

Table F-1 Summary of in situ Tests Used to Evaluate Liquefaction Resistance (based on Youd et al., 2001)

Feature	*SPT*	*CPT*	*VS*	*BPT*
Past measurements at liquefaction sites	Abundant	Abundant	Limited	Sparse
Type of stress-strain behavior influencing test	Partially drained, large strain	Drained, large strain	Small strain	Partially drained, large strain
Quality control and repeatability	Poor to good	Very good	Good	Poor
Detection of variability of soil deposits	Good for closely spaced tests	Very good	Fair	Fair
Soil types in which test is recommended	Non-gravel	Non-gravel	All	Primarily gravel
Soil sample retrieved	Yes	No*	No	No
Test measures index or engineering property	Index	Index	Engineering	Index

* CPT operator may have sampling equipment that can be used with the conventional CPT equipment to retrieve a small bulk sample

In boreholes, samples should be retrieved by driving an SPT split spoon sampler according to established procedures (ASTM D 6066-98 and ASTM D 1586; Martin and Lew, 1999; Youd et al., 2001), and the blowcount (N) from these tests should be recorded. The vertical spacing of the SPT is determined by site-specific needs but should be performed at intervals of not more than 5 feet or at significant stratigraphic changes. Care should be exercised when performing SPT tests in gravel deposits, where SPT N-values can be misleadingly high (Martin and Lew, 1999; Youd et al., 2001). The use of alternative split spoon samplers, such as the relatively large diameter California sampler, should be avoided for evaluations of penetration resistance. Cone penetration testing should be performed using an electronic cone and according to standard procedures (ASTM D 5778). At a minimum, measurements of CPT tip resistance (q_c), sleeve friction (f_s), and pore water pressure (u) should be made during the testing.

The depth of exploration for borings with SPT and CPT soundings should be sufficient to penetrate through soils potentially susceptible to liquefaction. Exploration may be ended at a depth at which the soil possesses a stiffness or consistency consistent with negligible liquefaction susceptibility, provided that the geologic environment is such that deeper exploration is unlikely to encounter additional liquefiable strata. Martin and Lew (1999) recommend that for liquefaction hazard studies, borings with SPT and CPT soundings should generally extend to depths of at least 50 feet below the ground surface.

The material samples retrieved during the subsurface exploration program can be used in laboratory testing to evaluate key soil index properties. Key tests that should be performed on each sample include water content (ASTM D 2937), gradation to establish fines content (i.e., percentage of soil by weight that passes a No. 200 sieve; ASTM D 422), and Atterberg Limits (ASTM D 4318).

F.3 Detailed Analysis of Liquefaction Triggering

When the soil at a site is judged to be potentially susceptible to liquefaction based on the screening analysis procedures presented above, more detailed analyses are needed to evaluate whether liquefaction was likely or unlikely to have actually occurred. The framework for liquefaction assessment (Figure F-1) is essentially an assessment of liquefaction triggering and liquefaction consequences. Liquefaction triggering procedures are the first step in the liquefaction assessment framework. The most commonly used techniques are based on case histories of liquefaction and non-liquefaction during past earthquakes and were pioneered by Seed and Idriss (1971) and Seed et al. (1985). Youd et al. (2001, 2003) updated several factors in the technique pioneered by H. Bolton Seed and was the last community consensus where a single procedure was accepted for estimating liquefaction triggering. It has since been recognized that the use of multiple liquefaction triggering procedures will provide a more robust analysis, with a better understanding of the range of estimates that may be obtained from these different models. Updated simplified procedures are available for both SPT data (Cetin et al., 2018; Boulanger and Idriss, 2014) and CPT data (Moss et al., 2006; Boulanger et al., 2016; Robertson, 2009). These procedures are based on larger and more carefully screened databases than those reflected in Youd et al. (2001). In addition, the Seed et al. (2003) procedures introduced curves for probability of liquefaction instead of single lines separating the liquefaction and non-liquefaction spaces.

Potential users of the new procedures are cautioned that such procedures must be used in their entirety, meaning that components of the new and old procedures should not be combined (i.e., values of a parameter from one method should not be corrected using values from a different method).

The “simplified procedures” described in the above references entail characterizing two parameters: *demand* imposed by cyclic loading during the seismic event and *capacity* of the soil to resist liquefaction. At depths where the demand exceeds the capacity, liquefaction was likely to have occurred. This is quantified by a factor of safety as follows:

$$FS = \frac{CRR}{CSR} \tag{F-1}$$

where *CRR* represents the cyclic resistance ratio (i.e., the soil resistance to liquefaction), and *CSR* represents the cyclic stress ratio (i.e., the stress demand placed on the soil during the earthquake). A condition of $FS < 1$ in a susceptible soil type suggests that liquefaction was likely to have been triggered, whereas $FS > 1$ implies that liquefaction was unlikely. In practice, recognizing the uncertainty involved in estimation of these parameters, a slightly higher FS may be used in the triggering assessment (e.g., FS > 1.1–1.3). Both *CRR* and *CSR* are dimensionless parameters, as they represent the ratio of seismic shear stress to effective normal stress prior to the onset of shaking. The evaluation of each of the terms in Equation F-1 is the subject of the following sections.

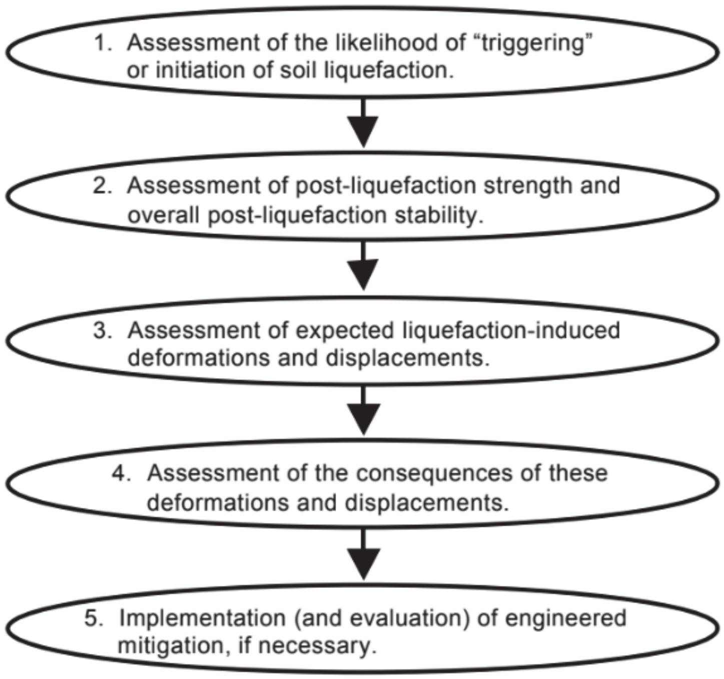


Figure F-1 Liquefaction assessment framework (Seed et al., 2003).

It is important to note that the case histories underlying the simplified procedures are based on post-earthquake observations of *surface manifestations* of liquefaction (e.g., ejecta, sand boils, cracking), implicitly pairing the estimation of liquefaction triggering with liquefaction consequences. The existing case histories are also largely biased toward cleaner sand sites. Application of the simplified procedures to sites with subsurface conditions that differ from the underlying case histories should be recognized as an extrapolation of those simplified procedures. Recent research has also recognized the importance of understanding system response at a site (Cubrinovski et al., 2019), working toward a more holistic evaluation of potential liquefaction and consequences. These efforts have focused on the impacts of multiple effects at a site (e.g., interlayering of liquefiable and non-liquefiable soils, partial saturation, pore pressure generation patterns) that may contribute to the suppression of surface manifestations of liquefaction that would be anticipated based on assessment of a one-dimensional soil column using the simplified procedures.

F.3.1 Cyclic Resistance Ratio (CRR)

The evaluation of *CRR* for level ground sites is based on three principal factors: (1) soil penetration resistance (measured using techniques described in Section F.2); (2) magnitude of the earthquake; and (3) in situ effective stress in the liquefaction-susceptible soil layers.

The soil penetration resistance values used for evaluation of *CRR* generally require correction for procedural factors, overburden effects, and for the effects of fines. The manner by which these corrections should be made are described in detail in the updated simplified liquefaction triggering procedures referenced above. The procedural corrections are necessary for SPT *N*-values but not for CPT. The procedure-corrected *N*-values represent 60% efficiency of the SPT driving process and hence are referred to as N_{60}. Overburden corrections involve adjusting the procedure-corrected penetration resistance values to an effective overburden pressure of 100 kPa (e.g., producing blow count $(N_1)_{60}$). Fines corrections are intended to increase penetration resistance measures that are artificially low relative to clean sands because of the lubricating effects of fines. The necessary corrections are provided in the reference for each simplified procedure. The resulting penetration resistance values for SPT and CPT are referred to as $(N_1)_{60\text{-}cs}$ and q_{c1N}, respectively. Similar corrections for BPT and V_s are discussed in Youd et al. (2001).

Once the corrected penetration resistance measure has been evaluated for the susceptible soil layers at a site, the *CRR* values that would apply for magnitude 7.5 earthquakes and shallow depths (corresponding to vertical effective stress σ'_v < 100 kPa) can be evaluated using the simplified liquefaction triggering procedures. Because of the magnitude and overburden pressure constraints on the *CRR* parameter estimated for a magnitude 7.5 earthquake and shallow depth, it is referred to here as *CRR**.

A correction for magnitude (*M*) is necessary because the number of cycles of earthquake shaking is strongly dependent on earthquake magnitude, and liquefaction resistance for a given amplitude of shaking is significantly dependent on the number of cycles. Accordingly, all other factors being equal, *CRR* will increase relative to *CRR** for $M < 7.5$ and decrease for $M > 7.5$. The resulting multiplicative correction factor is denoted in this document as C_M and referred to in the simplified procedures by other terms, such as “MSF” or “DWF.”

Magnitude scaling factors were proposed by Liu et al. (2001) that are distant-dependent and have a defined level of uncertainty. These C_M values are based on empirical regression equations for the equivalent number of stress cycles during earthquakes. C_M values that are dependent only on magnitude were also presented by Seed et al. (2001) and Idriss (1999). These curves are generally similar to the Liu et al. (2001) curves and are near the lower end of the range given by Youd et al. (2001).

A correction for overburden stress is necessary because: (1) the case histories of liquefaction and non-liquefaction that populate the data sets used to develop the simplified procedures are principally from shallow soil sites, and hence represent low overburden stress conditions (σ'_v < 100 kPa); and (2) soil resistance to liquefaction decreases with increasing σ'_v. Accordingly, *CRR** values must be decreased for deep soil layers with σ'_v > 100 kPa. The resulting multiplicative correction factor is denoted K_σ.

Overburden correction factors were proposed by Boulanger (2003) and Seed et al. (2001, 2003) that vary continuously with σ'_v (instead of being one for σ'_v < 100 kPa and decreasing for σ'_v > 100 kPa). The most recent iterations of correction factors are provided in the references for the updated simplified procedures, described above.

After evaluation of correction factors C_M and K_σ, the value of CRR for use in Equation F-1 can then be evaluated as follows:

$$CRR = CRR \times C_M K_\sigma \tag{F-2}$$

The correction factors for each simplified liquefaction triggering procedure are provided in the reference for that procedure.

It should be noted that liquefaction assessment continues to be a topic of ongoing research, with recent developments including new methods for liquefaction triggering, settlement evaluation, probabilistic considerations, and an increasing range of soil types for which the methods are applicable. The methods and references discussed in this document represent the standard of practice at the time of writing; however, the geotechnical consultant should maintain a proactive approach in staying apprised of advances in the field.

Additional procedures for CRR evaluation based on BPT and V_s data are presented in Youd et al. (2001). All of the CRR evaluation procedures presented here and in the cited references apply for level ground sites. Corrections for non-level ground sites are presented in Youd et al. (2001). Finally, it should be noted for completeness that procedures for CRR evaluation based on laboratory testing are used in rare instances; such procedures are discussed in Kramer (1996).

F.3.2 Cyclic Stress Ratio (CSR)

The cyclic stress ratio is calculated by the following formula originally developed by Seed and Idriss (1971):

$$CSR = 0.65\left(\frac{a_{\max}}{g}\right)\left(\frac{\sigma_{v0}}{\sigma'_{v0}}\right) r_d \tag{F-3}$$

where:

$a_{\max}$ = peak horizontal acceleration at ground surface

g = acceleration of gravity

σ'_{v0} = effective vertical overburden stress

σ_{v0} = total vertical overburden stress

r_d = stress reduction factor

Peak ground acceleration (PGA) contour maps are typically available from the USGS in the weeks following an earthquake and can be used to estimate PGA at a site. Youd et al. (2001) and the updated simplified methods provide guidance for selection of stress reduction coefficient, r_d. It should be noted that the procedures for evaluation of r_d are appropriate for use within their respective liquefaction triggering analysis procedures (i.e., the r_d term from one procedure should not be applied in a different procedure).

Recent advances in liquefaction assessment have incorporated alternative seismic demand parameters, such as the use of Cumulative Absolute Velocity (CAV) for building-adjacent liquefaction settlement estimates (Bray and Macedo, 2017), when those parameters provide an improved estimate (relative to PGA) of the effects under consideration.

F.4 Consequences of Liquefaction

The consequences of liquefaction may include settlement, cracking from ground oscillations, lateral spreading, instability of slopes and retaining walls, and instability and settlement of foundations. Analysis procedures for evaluation of these effects are generally much less maturely developed than those for liquefaction triggering. The discussion that follows divides the broad subject of liquefaction consequence into sections on post-liquefaction reconsolidation settlement, stability problems, and lateral spreading.

F.4.1 Post-Liquefaction Reconsolidation Settlement

If a soil stratum liquefies, the high excess pore pressures will dissipate over a period of time that may last from minutes to days (a process of classical consolidation). The amount of time required for the settlement to occur depends on several factors including hydraulic conductivity and compressibility of the soil and length of the drainage path. Post-liquefaction reconsolidation of soil may result in damage to structures and flatwork if it is manifest at the surface as differential settlement. This liquefaction induced damage will occur only within the period of time of post-liquefaction reconsolidation of soil; once the dissipation of excess pore pressures induced from the earthquake are complete, the soil resumes its aging process and damage induced from liquefaction is complete.

In cases of incomplete liquefaction (i.e., pore pressure is less than effective stress), the dissipation of pore pressure generated in the soil deposits may result in small amounts of settlement and typically will be very small and insignificant for most structures (Tokimatsu and Seed, 1987).

Procedures outlined by Tokimatsu and Seed (1987) and Ishihara and Yoshimine (1992) are widely used for estimating liquefaction induced settlements for level ground sites. Guidelines on the implementation on these procedures are provided by Martin and Lew (1999). Zhang et al. (2002) outlines a procedure for settlement estimates using CPT data.

Care should be exercised in implementing the above analysis procedures, particularly for the following conditions:

Soils that are not clean sands. When the soils are silty sands or silts, corrections may be applied to the SPT or CPT data to correct for the presence of fines. These correction factors are generally taken as the same values used for triggering analysis, as discussed above. When fine-grained soils or sands with appreciable fines are present and satisfy the susceptibility criteria listed in Section F.1, the correction factors no longer apply and cyclic laboratory testing may be required to evaluate their possible settlement contribution.

Layered soil deposits and surface disruption. Calculation of settlements of layered soil deposits requires consideration of the liquefaction susceptibility of each layer. Only layers that contain soils with a liquefiable material type and which were likely to generate significant pore pressure should be considered in settlement calculations. If a liquefiable stratum occurs beneath a non-liquefiable layer, the potential for the liquefaction effects to be manifest at the ground surface should be assessed using the guidelines of Ishihara (1985), which were later updated by Youd and Garris (1995) (see also Figure 5-6, which includes boundary curves for surface manifestation of liquefaction at different levels of peak ground acceleration). Care should be employed when using Figure 5-6, because it is not applicable to sites where flow failures or lateral spreading have occurred and also may not be applicable to soils with significant fines.

Differential settlements. Settlements calculated from a Tokimatsu and Seed (1987) or Ishihara and Yoshimine (1992) analysis are total settlements and must be converted to differential settlements in order to assess their damage potential. Preferably, when there is sufficient subsurface data, this calculation is made directly from subsurface results (i.e., differential settlements are calculated based on data from two or more subsurface investigation locations) that directly capture the variability of the subsurface conditions. When this data is not available, CDMG (1997) recommends a rule of thumb for differential settlements being 2/3 of the total settlements. Martin and Lew (1999) make an argument for less than 1/2 of the total settlements under uniform conditions at a site with deep sediments and 1/2 to 2/3 of the total settlement when subsurface conditions vary significantly laterally or vertically across a site. Sound engineering judgment should be used when estimating differential settlements from total settlement calculations.

F.4.2 Stability Problems

The most critical step in stability problems involving liquefied soil is the estimation of undrained residual strength (S_r), which is generally the strength parameter that should be used in stability calculations involving liquefied soils in slopes, foundations, and retaining walls. Parameter S_r represents the large-strain (residual) soil strength in liquefied soil zones. Procedures for estimation of S_r as a function of penetration resistance (CPT or SPT) have been developed by Olsen and Stark (2002) and Seed and Harder (1990) (SPT-only) based on back-analyses of flow failure case histories. The procedure by Olsen and Stark (2002) correlates S_r/σ_{v0}' to SPT or CPT penetration resistance without a fines correction; the procedure by Seed and Harder (1990) correlates S_r to SPT penetration resistance with a fines correction (the same clean sands correction factor used for liquefaction triggering can be used for estimation of S_r values, Seed et al., 2001). The correlation relationships for S_r values are found in Figure F-2.

For application purposes, engineers are encouraged to evaluate S_r by both techniques. When the procedures provide significantly different estimates of S_r, engineers must exercise judgment in selection of an appropriate value for back-analysis and design. While much work remains to be done on this issue, this document recommends giving more weight to the S_r estimate from Seed and Harder (1990), as consensus has not been reached regarding the use of S_r estimation procedures that are based on effective stress normalization. However, one issue to be especially careful of is to ensure that S_r values are lower than static shear strengths—this will not be an issue with the Stark and Olsen procedure but could be with the Seed and Harder (1990) procedure for shallow failures. For this reason, the Olsen and Stark (2002) procedure may be preferable for situations involving very shallow stability failures.

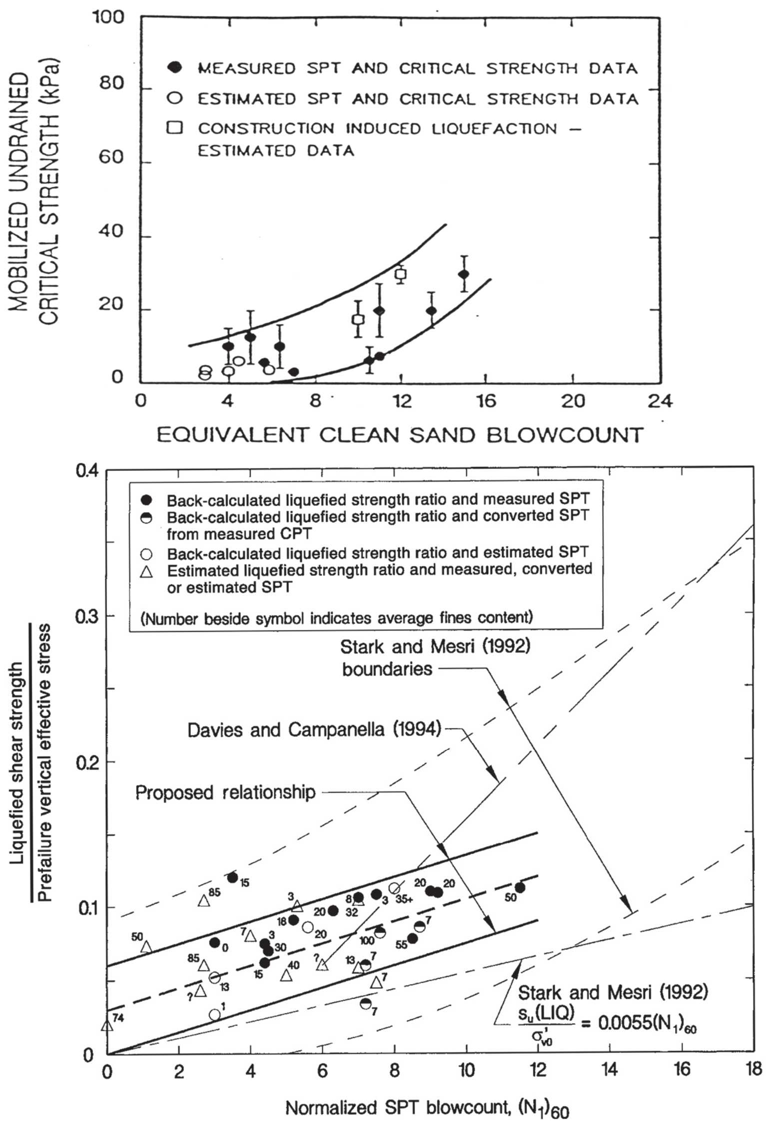


Figure F-2 Relationships for estimation of post-liquefaction residual strength (*Sr*) from SPT blow count (top: Seed and Harder, 1990; bottom: Olsen and Stark, 2002).

Once S_r values in the liquefiable soil have been evaluated, static stability analyses of the system under consideration (slope, foundation, retaining wall) should be performed. If the system is unstable with these strengths, a flow failure would be expected. If a flow failure did not occur (such a failure would usually be obvious from reconnaissance activities), the S_r values should be adjusted upward accordingly. If the system is statically stable in stability analyses performed with S_r values, the liquefaction problem was one of cyclic mobility, and shear deformations would have been confined to the time period of strong earthquake shaking. Analysis of expected displacement of slopes and retaining walls under these conditions are covered in Appendix H and Appendix I, respectively. Analysis procedures for foundation displacement under such conditions were not available at the time of writing this document. Development of such procedures is a research need.

Recent developments in residual shear strength estimation include Kramer and Wang (2015), Weber et al. (2015), and Idriss and Boulanger (2015). These methods provide useful methodologies for gaining insight into the behavior of a site, in addition to the methods discussed earlier in this section.

F.4.3 Lateral Spreading

Flow slides or cyclic mobility on very gently sloping ground or on nearly flat ground adjacent to drainage or stream channels or bodies of water can produce permanent lateral ground displacements in the direction of the driving static shear stress. The occurrence of these displacements is referred to as lateral spreading. Lateral spreading may produce conspicuous surface manifestations of permanent ground deformation that, in a post-earthquake investigation, shifts the question from if permanent ground deformation occurred to what may be the appropriate repair and mitigation strategy. However, in some instances, it may be necessary to differentiate between permanent ground deformation modes or to estimate the magnitude of displacement from lateral spreading. This is generally accomplished using empirical approaches described below. It should be noted that those approaches are valid for moderate ground slopes (< ~6%) and for distances behind a free face ranging from approximately 5 to 100 times the free face height. Lateral ground movements closer to a free face or on steeper slopes are considered stability problems and are analyzed differently (see Section F.4.2).

Lateral spread displacements can be estimated using empirical models. One such model is based on multilinear regression of a large case history database. The current community consensus version of this approach is described in Youd et al. (2002). The method is applicable to sites with gently sloping ground (0.1%–6% slope) or ground with a free face (i.e., drainage or stream channels or bodies of water). The first step in applying this method is to determine if liquefaction has been triggered at the site according to the procedures described above. Provided liquefaction was triggered, the Youd et al. (2002) manuscript should be consulted for guidance regarding calculations of lateral spread displacements. Another promising procedure for calculation of lateral spread displacements involves the calculation of shear strain potential in each liquefiable layer within a soil profile down to a maximum depth of Z_{max}, where Z_{max} is the maximum depth below all the potentially liquefiable layers with a calculated FS < 2.0 (Zhang et al., 2004). Integration of the shear strains with depth provides a displacement index that is related to actual displacement for sloping ground or free-face conditions using empirical relationships. Details on this approach are presented in the Zhang et al. (2004).

Recent developments in lateral spreading case histories and assessment methods are available (Cubrinovski and Robinson, 2016; Franke and Kramer, 2014) but these have not reached widespread use or acceptance at the time of this writing. These developments are identified to offer additional tools to supplement the methods discussed in this appendix. As discussed for liquefaction assessment, lateral spreading assessment is a topic of ongoing research. The methods and references discussed in this document represent the standard of practice at the time of writing.

APPENDIX G

Seismic Compression

G.1 Introduction

Appendix G presents simplified procedures for estimating ground displacements from seismic compression.

An analysis of seismic compression for a site begins with an assessment of susceptibility. Susceptible soils include granular soils, silts, and low-plasticity clays. Highly plastic clays (i.e., where the plasticity index, PI > approximately 30) and soils with fines content greater than approximately 10% tend to have a low susceptibility to seismic compression (Yee et al., 2014). Soils at high confining stresses, and those with moderate degrees of saturation (approximately 30%–60%), which correspond to high levels of matric suction, have also been shown to have relatively lower susceptibility to seismic compression (Ghayoomi et al., 2013; Yee et al., 2014).

In the following, two simplified procedures for estimating ground displacements from seismic compression are presented. The procedures share three common steps: (1) estimation of shear strain amplitude within the soil mass from the peak acceleration at the ground surface and other seismological and site parameters; (2) estimation of volumetric strains within the soil based on soil density and water content, the shear strain amplitude, and the equivalent number of uniform strain cycles; and (3) integration of volumetric strains across the soil section to estimate settlement. One of the procedures presented is that of Tokimatsu and Seed (1987), which is strictly applicable only to clean sands (i.e., natural soil or fill). The second procedure was developed as part of research funded by the CUREE Earthquake Damage Assessment project (Stewart et al., 2004b) and is applicable to compacted fill soils. The procedure for compacted fills applies for a variety of soil fines contents and fines plasticity. An early version of the compacted fills procedure has been verified relative to three well-documented case histories by Stewart and Whang (2003). A similar model for volumetric strains in clean sands was developed by Duku et al. (2008) for clean sands and extended by Yee et al. (2014) to account for the fines content of soils.

In the following sections, analysis procedures intended strictly for fill soils are distinguished from those that can also be used for natural soils; associated subsurface exploration and laboratory testing approaches are also presented.

G.2 Subsurface Exploration and Laboratory Testing

The objectives of subsurface exploration and laboratory testing for seismic compression analyses are to: (1) evaluate the thickness of soil layers potentially susceptible to seismic compression across the site; (2) evaluate index properties such as Atterberg limits and fines content of soil materials; (3) estimate the in situ density and water content of soil materials; and (4) evaluate the depth to groundwater, if present.

Subsurface exploration may involve trenching, the drilling of boreholes, or cone penetration testing (CPT). CPT profiling should not be the sole method of site exploration. However, CPT profiling in conjunction with drilling and sampling can efficiently provide accurate information on subsurface stratigraphy. A sufficient number of exploration points (e.g., borings, CPTs, or trenches) should be used to reasonably evaluate variations in soil layer thicknesses across the site. Generally, this will require subsurface exploration at a minimum of three locations.

In situ densities can be evaluated with downhole in situ sand cone tests (ASTM D 2419) or through laboratory testing of samples retrieved in the field (ASTM D 2937). Samples to be used for such purposes should be disturbed to the least extent possible, as disturbance will change sample density. Disturbance is minimized by carefully hand-carving samples from trenches or downhole-logged boreholes or by using pushed thin-walled tube samples (e.g., Piston tube, Shelby tube). The driving of relatively thick-walled samplers (e.g., the Modified California sampler) can lead to biased estimates of soil density (Noorany, 1987) and should generally be avoided.

In addition to density, soil index tests that are useful in seismic compression analyses include the following:

- water content (ASTM D 2937)
- gradation (ASTM D 422 or ASTM D 1140)
- liquid limit and plastic limit (ASTM D 4318)

These tests should be performed using the cited ASTM standards. In addition, tests are required so that the soil density can be quantified in a relative sense, which involves the use of the relative density (D_r) parameter for clean sands and the relative compaction (RC) parameter for compacted soils with fines. Here clean sands are considered as those with a sufficiently low fines content that the fines do not affect the mechanical behavior of the soil. Clean sands are defined in ASTM D 2487 as those with fines content of less than 5% fines.

Relative compaction is measured using the Modified Proctor standard (ASTM D 1557). It is nearly impossible to obtain undisturbed samples for clean sands, and as a result, density measurements are invariably biased relative to in situ conditions. Accordingly, for such materials it is recommended that D_r be estimated using correlations with penetration resistance parameters, such as SPT N-value or CPT tip resistance. A number of these correlations are published in Kulhawy and Mayne (1990).

Water table depth can be established in boreholes that are left open for a sufficient period of time for groundwater levels to equilibrate or from CPTs configured with piezometers (if the CPT test is performed sufficiently slowly that piezometric heads are allowed to equilibrate). Water table depth is important, because seismic compression is only possible in unsaturated soils. Below the water table, the problem shifts to one of potential liquefaction (see Section 7.2).

G.3 Tokimatsu and Seed Procedure for Clean Sands

This method is applicable for clean sands, both natural soil and fill.

The original Tokimatsu and Seed (1987) analysis procedure is based on a simplified representation of the distribution of shear stress with depth in a one-dimensional soil column. If the soil column above a soil element at depth *h* behaves as a rigid body, and the ground surface peak horizontal acceleration is *PHA*, then the mass of soil above *h* would impose a maximum shear stress of:

$$\tau_{rigid,\max} = \frac{PHA}{g} \times \sigma_0 \tag{G-1}$$

where g = the acceleration due to gravity and σ_o = total overburden pressure at depth h.

Soil flexibility reduces the shear stress to values less than $\tau_{rigid,\max}$, as a result of vertical incoherence of ground motion. Seed and Idriss (1971) developed a simplified technique to estimate earthquake-induced cycle shear stresses at depth. They multiplied $\tau_{rigid,\max}$ by a stress reduction factor, r_d (which is the ratio of the actual shear stress at depth to the theoretical "rigid body" shear stress). A factor of 0.65 is then applied to reduce the peak cyclic shear stress, $\tau_{\max}$, to the effective cyclic stress, τ_{eff}, as:

$$\tau_{eff} = 0.65 \times \frac{PHA}{g} \times \sigma_0 \times r_d \tag{G-2}$$

Effective shear strain, γ_{eff}, is estimated from τ_{eff} using the effective shear modulus (G_{eff}), as follows:

$$\gamma_{eff} = \frac{\tau_{eff}}{G_{eff}} = \frac{\tau_{eff}}{G_{\max}\left(\dfrac{G_{eff}}{G_{\max}}\right)} \tag{G-3}$$

where $G_{\max}$ = small strain shear modulus. Combining Equation G-2 and Equation G-3 leads to:

$$\gamma_{eff} \frac{G_{eff}}{G_{\max}} = \frac{0.65 \times PHA \times \sigma_0 \times r_d}{g \times G_{\max}} \equiv P \tag{G-4}$$

The product $\gamma_{eff}\,(G_{eff}/G_{\max})$ in Equation G-4 can be readily translated to a shear strain amplitude γ_{eff} using published models for soil modulus reduction with increasing shear strain (i.e., models relating γ_{eff} to $G_{eff}/G_{\max}$). Tokimatsu and Seed (1987) recommended using the modulus reduction curves of Iwasaki et al. (1978), which depend on effective stress.

Having estimated γ_{eff} with the above procedure, volumetric strains at 15 cycles of shaking [$(\varepsilon_v)_{N=15}$] are estimated using an appropriate volumetric strain material model. These models relate $(\varepsilon_v)_{N=15}$ to γ_{eff}, and depend on soil D_r. Tokimatsu and Seed (1987) utilized the volumetric strain material model of Silver and Seed (1971), which are derived from laboratory simple shear testing of clean sands.

The values of $(\varepsilon_v)_{N=15}$ are adjusted to the volumetric strain (ε_v) for the actual number of strain cycles (N) using the factor $C_N = \varepsilon_v/(\varepsilon_v)_{N=15}$. Tokimatsu and Seed (1987) recommended using C_N relations for clean sand derived from testing by Silver and Seed (1971). Parameter N is a ground motion intensity measure (like *PHA*), and Tokimatsu and Seed (1987) recommended that it be estimated using an empirical relationship between magnitude (m) and N proposed by Seed et al. (1975).

The N-adjusted volumetric strain ε_v is typically considered to range by a factor of 0.5 to 2, where the upper range accounts for multi-directional shaking effects per the recommendations of Pyke et al. (1975). Hence, the final estimate of volumetric strain range at a point is represented by $0.5 \times C_N \times (\varepsilon_v)_{N=15}$ to $2 \times C_N \times (\varepsilon_v)_{N=15}$. These volumetric strains are then integrated over the depth of the soil column to calculate settlement.

G.4 Procedure for Compacted Fill Soils

This procedure follows the same basic steps as outlined above for the Tokimatsu and Seed (1987) procedure. However, the present procedure incorporates a number of significant developments since the earlier publication.

The procedure can be summarized as follows:

1. Sublayer the site for seismic compression analysis using relatively thin soil layers whose boundaries capture variations in material type and properties and capture significant variations of seismic demand (shear stress with depth).

2. For each sublayer, evaluate quantity P using Equation G-4, with stress reduction factor r_d evaluated based on the model of Seed et al. (2001) as follows:

$$z < 20 \text{ m: } r_d = \frac{\left[1 + a_1/a_2(z)\right]}{\left[1 + a_1/a_3\right]} \tag{G-5}$$

$$z > 20 \text{ m: } r_d = \frac{\left[1 + a_1/a_2(z=20)\right]}{\left[1 + a_1/a_3\right]} - 0.0046(z-20) \tag{G-6}$$

where:

$$a_1 = -23.013 - 2.949 PHA/g + 0.999m + 0.0053 V_{s-12} \tag{G-7}$$

$$a_2(z) = 16.258 + 0.201 e^{0.341(-z + 0.0785 V_{s-12} + 7.586)} \tag{G-8}$$

$$a_2\,(z = 20) \text{ is } a_2\,(z) \text{ with } z \text{ set to 20 m} \tag{G-9}$$

$$a_3 = 16.258 + 0.201 e^{0.341(0.0785 V_{s-12} + 7.586)} \tag{G-10}$$

z = depth in meters

m = earthquake magnitude

V_{s-12} = average shear wave velocity in upper 12 m of the site (in m/s)

3. Estimate the equivalent number of uniform strain cycles (N) based on earthquake magnitude and site-source distance using Figure G-1 or the following:

$$N = \frac{\left(\dfrac{\exp\left(b_1 + b_2\left(m - m^*\right)\right)}{10^{1.5m+16.05}}\right)^{-\frac{1}{3}}}{4.9\times10^{6}\beta} + Sc_1 + rc_2 \qquad \text{(G-11)}$$

where r is in km, $b_1 = 1.53$, $b_2 = 1.51$, $c_1 = 0.75$, $c_2 = 0.095$, $\beta = 3.2$, and $m^* = 5.8$. Parameter S is zero if rock or shallow soil (< 20 m) underlies the fill and 1 if > 20 m soil underlies the fill.

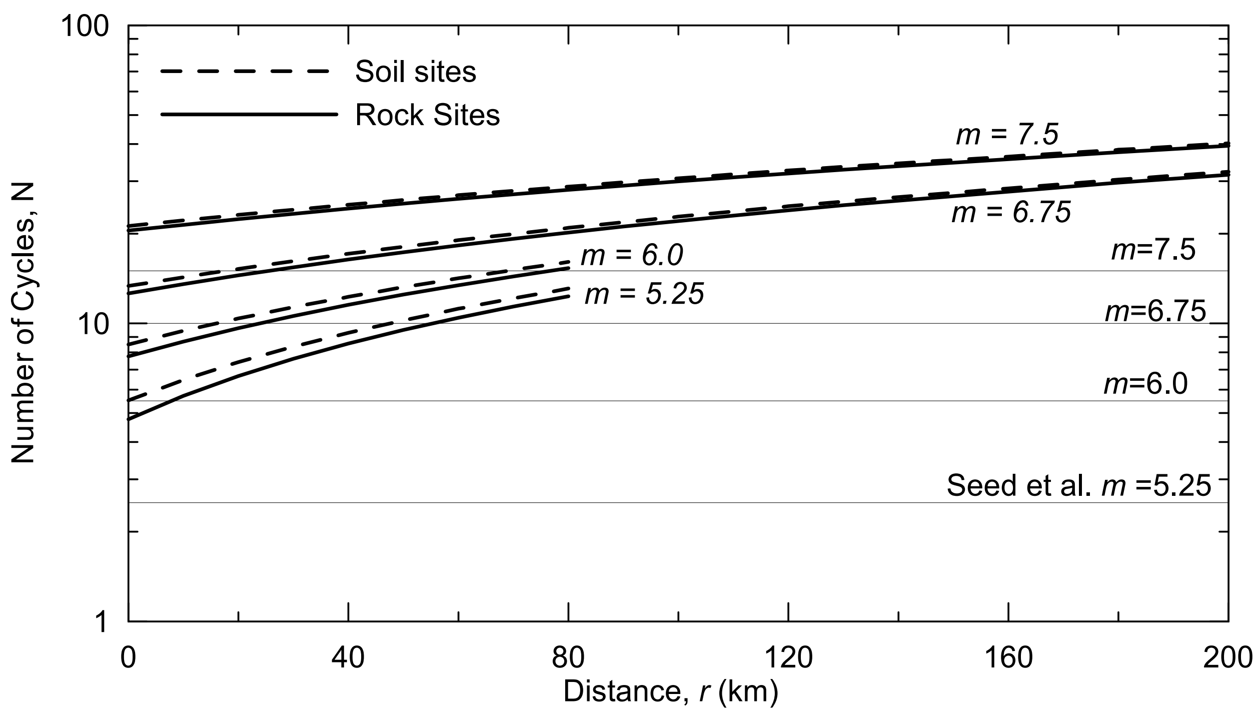


Figure G-1 Variation of median values of uniform straing cycles, N, with distance, r, and magnitude, m (Liu et al., 2001, along with recommendations of Seed et al., 1975).

4. Use an appropriate modulus reduction curve in conjunction with the P values from Step 2 to estimate shear strains in each sublayer. Tokimatsu and Seed (1987) recommended the use of modulus reduction curves for clean uniform sands by Iwasaki et al. (1978), which depend on effective stress. The model for modulus reduction by Darendeli and Stokoe (2001) is recommended for more general use, because it is based on a much larger suite of test results and incorporates effects of effective stress (σ'), soil plasticity (as represented by PI), and overconsolidation ratio (OCR). Figure G-2 shows a family of modulus reduction curves (based on the Darendeli and Stokoe, 2001 model) for varying PI and σ'(the effects of OCR are generally small, and the plots in Figure G-2 apply for OCR = 1, which is generally appropriate for fills at z >3m–6 m, Duncan et al., 1991). Note that the plots in Figure G-2 are formatted to directly estimate shear strain, γ, from the product $\gamma(G/G_{\max})$.

Pradel (1998) developed a fit to the Iwasaki et al. (1978) curves shown in Figure G-2 using the following equation:

$$\gamma = \frac{1+g_1 \times e^{g_2 P}}{1+g_1} P \times 100 \text{ (in \%)} \tag{G-12}$$

where P is the product computed in Equation G-4. The same regression equation is used for the Darendeli and Stokoe (2001) curves, with g_1 and g_2 related to soil type as follows:

PI ≈ 30: $g_1 = 4.0$ $g_2 = 1400$ (G-13)

PI ≈ 15: $g_1 = 0.194(\sigma'/p_a)^{0.265}$ $g_2 = 7490(\sigma'/p_a)^{-0.418}$ (G-14)

PI ≈ 0: $g_1 = 0.199(\sigma'/p_a)^{0.231}$ $g_2 = 10850(\sigma'/p_a)^{-0.410}$ (G-15)

where p_a = 101.3 kPa. Shear strains for intermediate PIs can be interpolated.

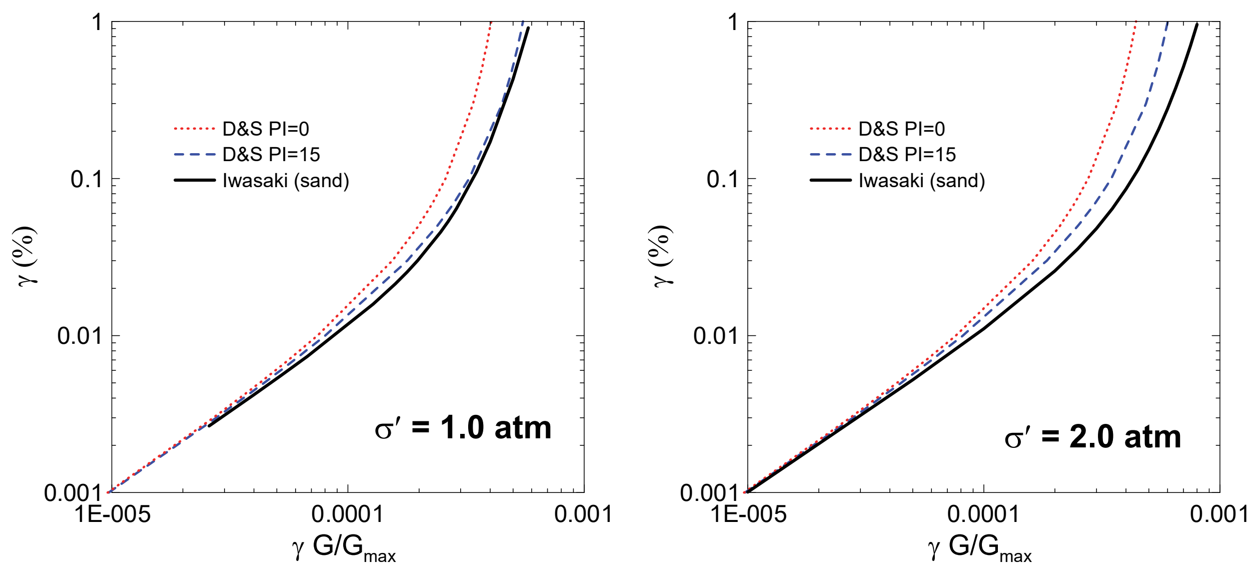


Figure G-2 Modulus reduction curves re-expressed in format for estimation of shear strain amplitude, showing effects of effective overburden stress and soil plasticity (Iwasaki, 1978; Darendeli and Stokoe, 2001).

5. Employ an appropriate volumetric strain material model to estimate volumetric strains within each sublayer. A volumetric strain material model is defined as a relationship between: (1) cyclic shear strain amplitude, γ_c, and $(\varepsilon_v)_{N=15}$; and (2) C_N and N. Volumetric strain material models based on laboratory simple shear testing are discussed in the following sub-section. The range of volumetric strains are $0.5 \times C_N \times (\varepsilon_v)_{N=15}$ to $2 \times C_N \times (\varepsilon_v)_{N=15}$.

6. Evaluate settlement by summing the product of volumetric strains within each sublayer and the corresponding sublayer thickness. The depth range over which these volumetric strains should be integrated should be sufficiently large that significant causes of potential differential settlement are captured. If there are significant lateral variations in the thickness of materials subject to seismic

compression, integrations should be performed at multiple locations using the full thickness of susceptible layers. If the thickness of susceptible layers is consistent across the site, only volumetric strains occurring relatively close to the ground surface are likely to produce significant differential settlement. Accordingly, it is recommended that the integration be carried to a depth corresponding to the approximate width of the surface improvements that might be affected by differential settlement. Differential settlements can then be estimated from total settlements using standard empirical "rules of thumb" (e.g., Grant et al., 1974).

G.5 Volumetric Strain Material Models

A key step in either the Tokimatsu and Seed (1987) procedure or the compacted fills procedure is the evaluation of volumetric strains using a volumetric strain material model, which is defined as a relationship between: (1) cyclic shear strain amplitude, γ_c, and $(\varepsilon_v)_{N=15}$; and (2) C_N and N.

Tokimatsu and Seed (1987) recommended the use of volumetric strain material models that were derived from cyclic simple shear testing of clean sands by Silver and Seed (1971). More recent simple shear testing programs have re-examined these relationships for clean sand and have developed a significant database of test results for fill soils containing fines. Because of different material performance characteristics, volumetric strain material models from these tests are presented for three categories of material characteristics: (1) clean sands; (2) sandy soils containing non-plastic silts; and (3) sandy soils containing fines with variable levels of plasticity. Additional information on the testing reported below can be found in Stewart et al. (2004b) and Whang et al. (2004).

G.5.1 Clean Sands

A series of fourteen different clean sand materials were tested that span a wide range of properties, such as material gradation, particle size, and particle shape. The tests were performed under drained conditions with a vertical stress of 101.3 kPa. Samples were subject to a sinusoidal loading frequency of 1.0 Hz and shear strain amplitudes varying from γ_c = 0.1% to 1%. The relationship between γ_c and vertical strain at 15 cycles $(\varepsilon_v)_{N=15}$ from the tests are shown in Figure G-3 for D_r = 60%. Also shown in Figure G-3 is a power-law curve fit through the data, plus and minus one standard deviation on the fit, and a line fit through the Silver and Seed (1971) data. The power law fit is described by the following equation:

$$\gamma_c > \gamma_{tv}:\ (\varepsilon_v)_{N=15} = a(\gamma_c - \gamma_{tv})^b \qquad \text{(G-16a)}$$

$$\gamma_c < \gamma_{tv}:\ (\varepsilon_v)_{N=15} = 0 \qquad \text{(G-16b)}$$

where a, b, and γ_{tv} are dependent on soil composition and compaction condition and are estimated from the laboratory testing. No trends in the $(\varepsilon_v)_{N=15}$-γ_c relationship were found relative to sand compositional factors, although the collective results provide insight into the variability of $(\varepsilon_v)_{N=15}$ for a given γ_c. Residuals of the power-law fit are approximately normally distributed, hence the variability is characterized by a coefficient of variation of 0.37. Note that the median fit in Figure G-3 is generally consistent with the Silver and Seed (1971) results.

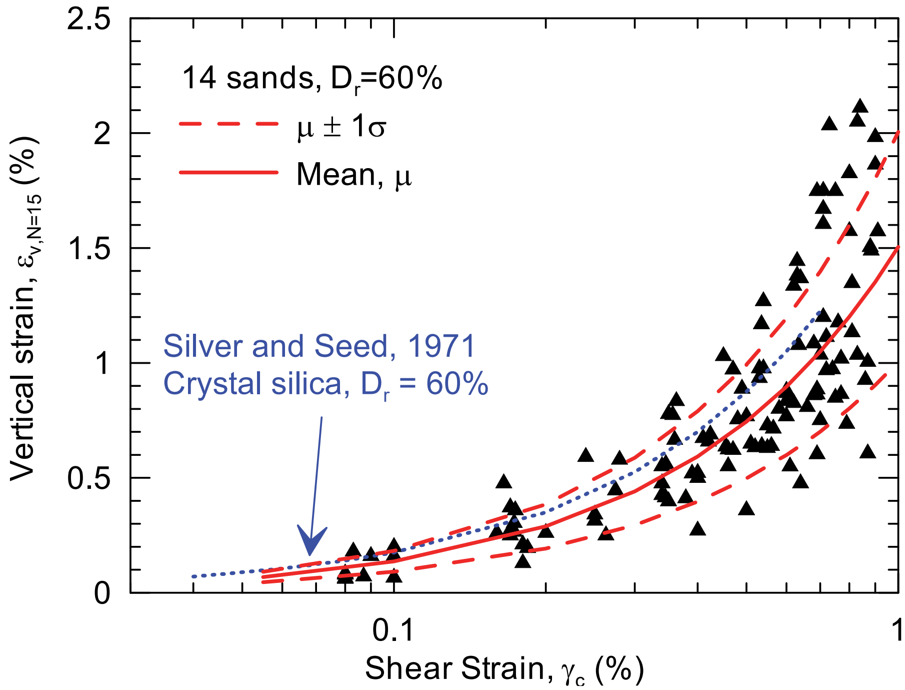


Figure G-3 Volumetric strains for clean sands at D_r = 60% (Stewart et al., 2004b).

Figure G-4 shows results of tests on two representative sand materials at the relative density levels of D_r = 45% and 80% along with the D_r = 60% fit curve from Figure G-3. The test data from these two sands at D_r = 60% (not shown) is consistent with the fit curve for all tested sands shown in Figure G-3 and Figure G-4. The median curves in Figure G-4, or the corresponding regression coefficients shown in the figure for use with Equation G-16, can be used to estimate $(\varepsilon_v)_{N=15}$ for clean sands. Note that in Figure G-4, the parameter γ_{tv} is given as 0.01%, which can be compared with the recommended range of approximately 0.01%–0.02% by Hsu and Vucetic (2004). Volumetric strains derived using Equation G-16 and the coefficients in Figure G-4 should be considered applicable for $\gamma_c < 1.0$ %.

The C_N-N data from the CSS tests are nearly log-linear and hence can be described by the expression:

$$C_N = R \times \ln(N) + c \tag{G-17}$$

All soils must have $C_N = 1$ at N=15, which implies that intercept parameter $c = 1 - \ln(15)\times R$. Consequently, the C_N to N relationship for a given soil is fully described by slope parameter R. Values of R for sands are shown in Figure G-5. Parametric studies indicate that R does not vary with γ_c or D_r, and for practical purposes, the mean value of R = 0.33 can be used. The distribution of the data around the mean is approximately normal with a standard deviation of 0.04.

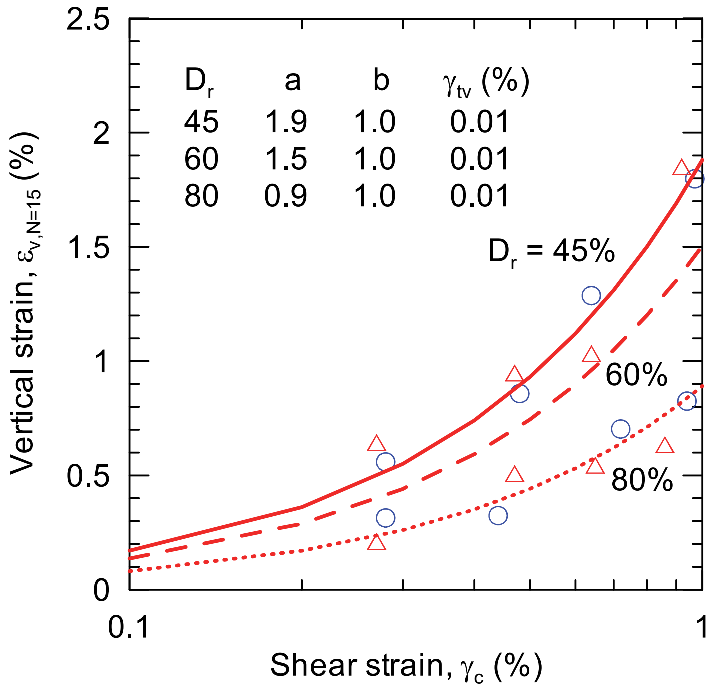


Figure G-4 Volumetric strain models for clean sands at D_r = 45%, 60%, and 80% (Stewart et al., 2004b).

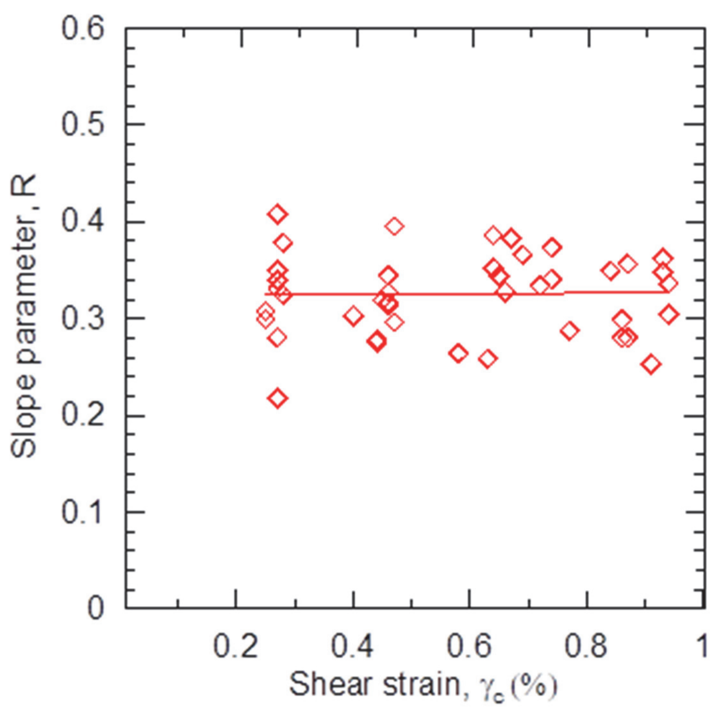
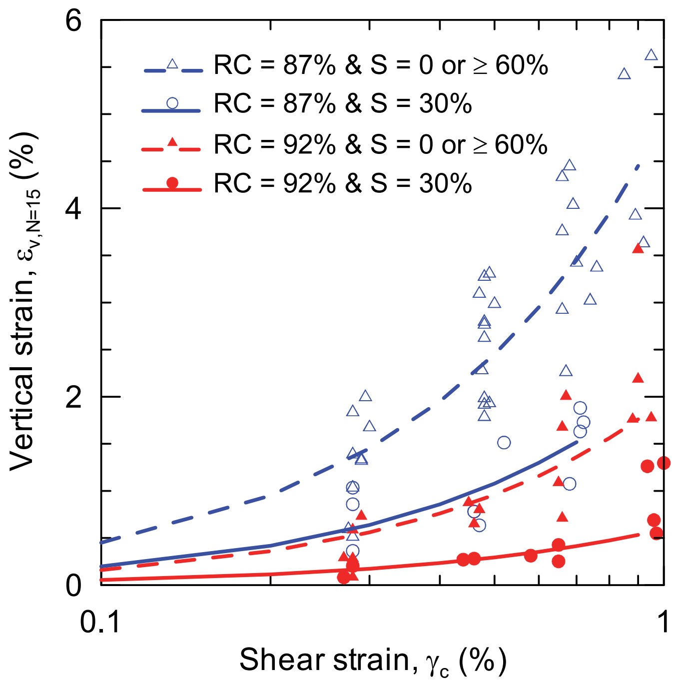


Figure G-5 Values of slope parameter *R* for clean sands (Stewart et al., 2004b).

G.5.2 *Soils with Non-Plastic Fines*

A series of eight different silty sand materials were tested that span a range of fines contents, as-compacted relative compaction levels (relative to modified Proctor densities), and as-compacted degrees of saturation. The silt materials added to the sands consist predominantly of quartz minerals that are truly non-plastic (i.e., unmeasurable plastic limit). Simple shear tests were performed using the same protocols as for the sands.

The γ_c-$(\varepsilon_v)_{N=15}$ test data from soil mixtures with 50% sand and 50% silt by weight are shown in Figure G-6. These materials have unmeasurable liquid limits based on ASTM procedures, but LL is estimated as < ~ 17. Note that there are significant effects of relative compaction (*RC*) and of *S* for these materials. As expected, the effect of increasing *RC* is to decrease $(\varepsilon_v)_{N=15}$. The effect of intermediate $S \approx 30\%$ is to decrease $(\varepsilon_v)_{N=15}$ relative to values for dry ($S = 0$) and high saturation ($S \geq 60\%$) conditions, which produce similar $(\varepsilon_v)_{N=15}$ and hence are grouped together in Figure G-6. Tests were also performed at intermediate fines contents between 0% and 50%. Results of these tests indicate that $(\varepsilon_v)_{N=15}$ for non-plastic fines content between 0% and 50% can be estimated by interpolating between the results from Figure G-4 and Figure G-6.



RC	S	a	b	τ_{hv} (%)
87	30	2.2	1.0	0.01
	0, ≥ 60	5.0	1.0	0.01
92	30	0.6	1.0	0.01
	0, ≥ 60	2.0	1.0	0.01

Figure G-6 Volumetric strains for 50-50 sand-silt mixtures, where RC 87 ↔ D_r 60 and RC 92 ↔ D_r 80 (Stewart et al., 2004b).

The C_N-N data from the CSS tests are nearly log-linear as described by Equation G-17. As with the data for sands, parameter *R* was found to be independent of other parameters (e.g., as shown in Figure G-7). The median value of *R* for non-plastic, silty sands is 0.36 with a standard deviation of 0.04.

Note by comparing Figure G-3, Figure G-4, and Figure G-6 that the effect of the silt is to increase the soil's seismic compression susceptibility relative to clean sand. As described further below, this effect is not observed for more "natural" fine-grained soil materials, which usually have more complex mineralogy involving some clay minerals (and hence measurable plasticity). Because the fines mineralogy used in the soils described above seldom occurs in nature, practical application of the results presented in Figure G-6 and Figure G-7 is likely limited.

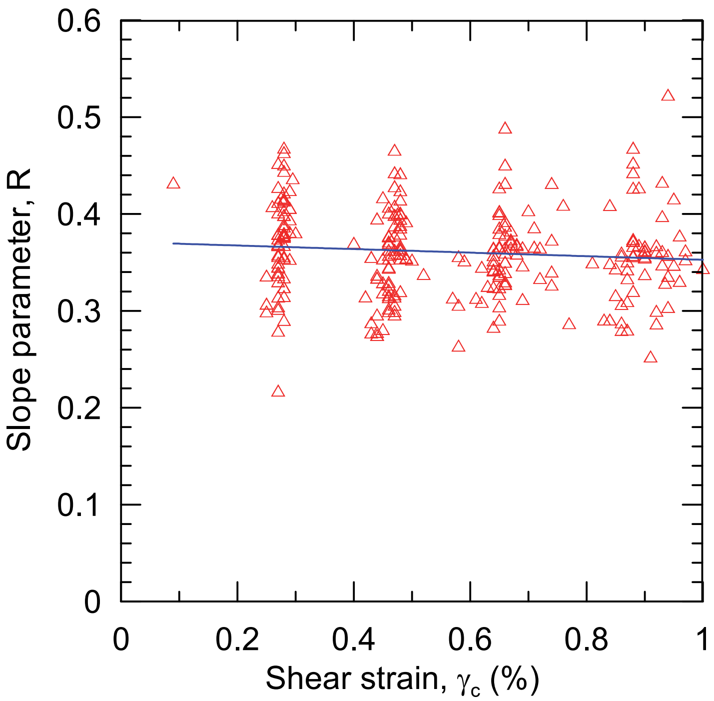


Figure G-7 Slope parameter R for silty sand test results shown in Figure G-6 (Stewart et al., 2004b).

G.5.3 Soils with Variable-Plasticity Fines

Suites of simple shear tests have been performed on six soil materials with large fines content (approximately 50%) and levels of soil plasticity (PI) varying from values of 2 to 27. The results enable the development of volumetric strain material models for soils with variable plasticity.

As shown in Figure G-8, low plasticity materials (tested material has PI = 2, LL = 27) exhibit *RC*- and *S*-dependent behavior similar to the non-plastic silts described above. Vertical strain $(\varepsilon_v)_{N=15}$ decreases with increasing *RC*. There is no obvious effect of saturation across the tested range of S = 55%–98%, which is similar to the behavior of the non-plastic silty sands described above. A significant difference from the non-plastic silty sand results is that overall strain levels are slightly reduced from those for clean sands (non-plastic silts had larger strains than clean sands). Parameter *R*, which describes the C_N-*N* relationship, is effectively independent of other parameters (Figure G-9) and has a mean value of about 0.32.

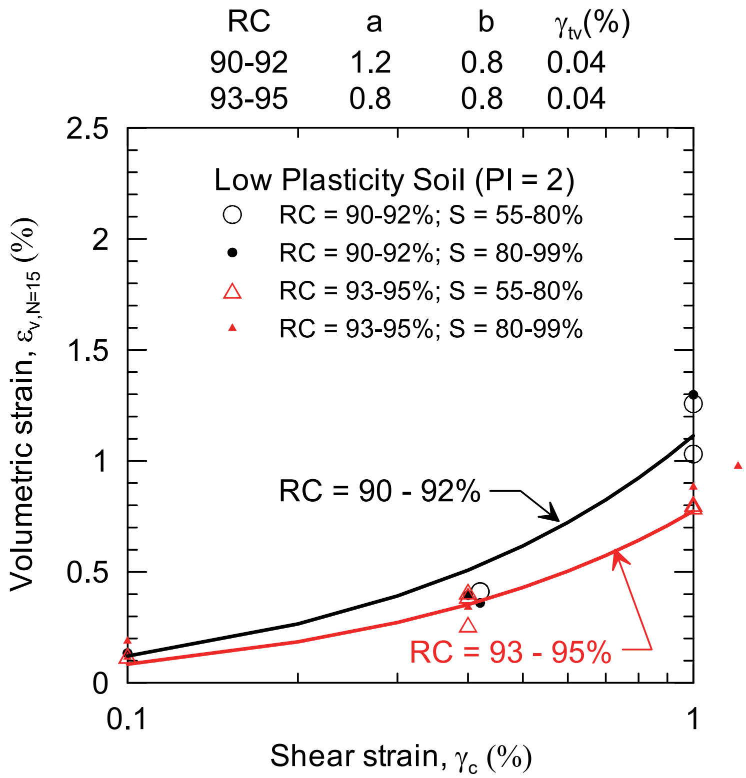


Figure G-8 Volumetric strains for low plasticity fine-grained soil, where PI = 2 and LL = 27 (Stewart et al., 2004b).

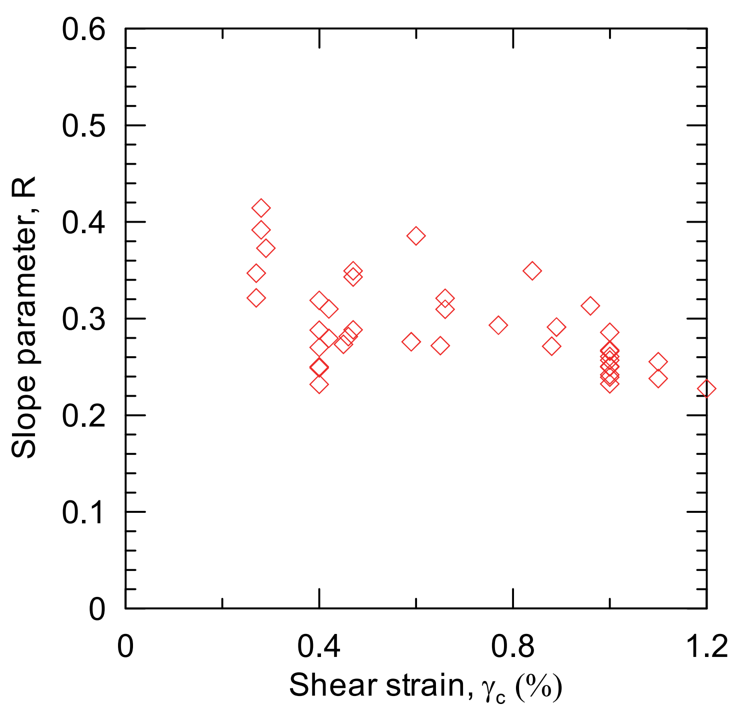
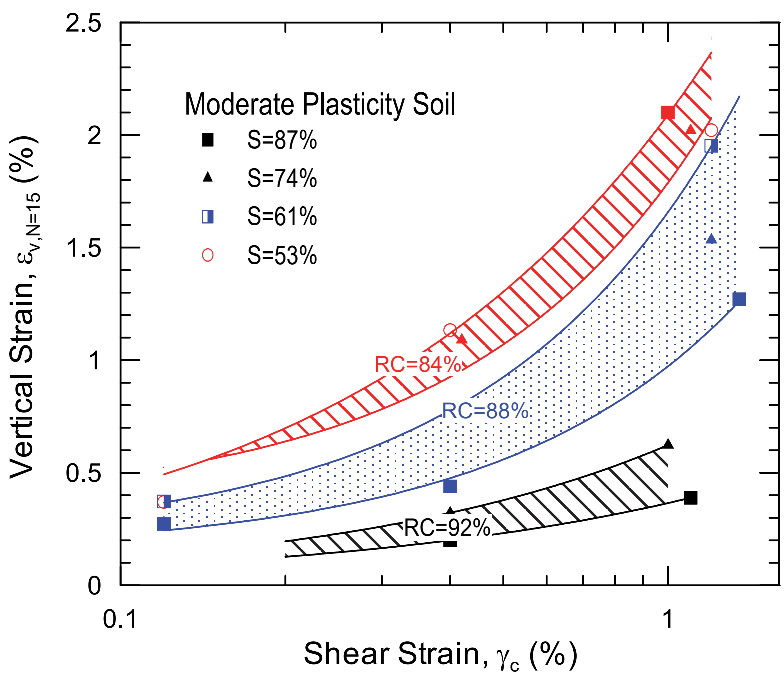


Figure G-9 Values of slope parameter R test results shown in Figure G-8 (Stewart et al., 2004b).

As shown in Figure G-10, moderate plasticity materials (tested material has PI = 15, LL = 33) exhibit *RC*- and *S*-dependent behavior, although the trends are different from those observed in non-plastic and low-plasticity materials. Vertical strain $(\varepsilon_v)_{N=15}$ decreases with increasing *RC* and increasing saturation (*S*). The variability with *S* is due to variations in the soil macro-structure, which consists of clods at low *S* (less than ~ 80%, which corresponds approximately with the line of optimums) and a near continuum for higher *S* (materials compacted wet of the line of optimums). There is no significant *S*-dependence of $(\varepsilon_v)_{N=15}$ at *RC* = 84%, because a clod structure occurs at all saturation levels at these low densities. Parameter *R* is effectively independent of shear strain (Figure G-11) and has a mean value of about 0.34.

As shown in Figure G-12, high plasticity materials (tested material has PI = 27, LL = 47) at low *RC* ≈ 87% exhibit *S*-dependent behavior similar to those at intermediate plasticity. No effect of saturation was observed at higher *RC* ≈ 92%, as the clod structure was largely broken down during compaction at those relatively high densities. No measurable effect of density (between *RC* ≈ 87%–92%) was observed for materials at high saturation. In general, vertical strains $(\varepsilon_v)_{N=15}$ for these relatively plastic materials are significantly lower (by about a factor of two) than those presented previously in Figure G-10. Parameter *R* from the test data is shown in Figure G-13, and has a mean value of 0.25 and standard deviation 0.04.



RC	S	a	b	γ_{tv}(%)
84	any	2.0	0.65	0.03
88	60	1.7	0.75	0.02
88	90	1.0	0.75	0.02
92	70	0.65	0.75	0.02
92	90	0.35	0.75	0.02

Figure G-10 Volumetric strains for moderate plasticity fine-grained soil, where PI = 15 and LL = 33 (Stewart et al., 2004b).

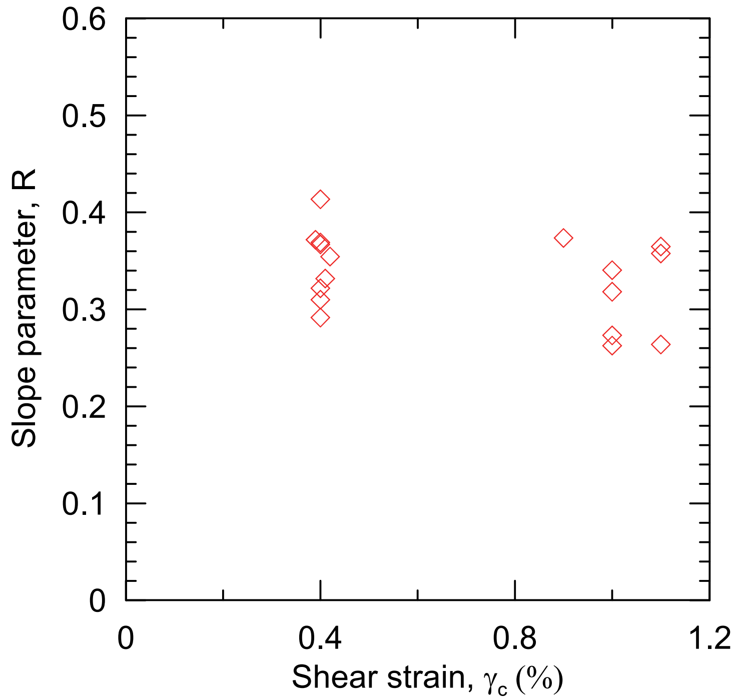


Figure G-11 Values of slope parameter R test results shown in Figure G-10 (Stewart et al., 2004b).

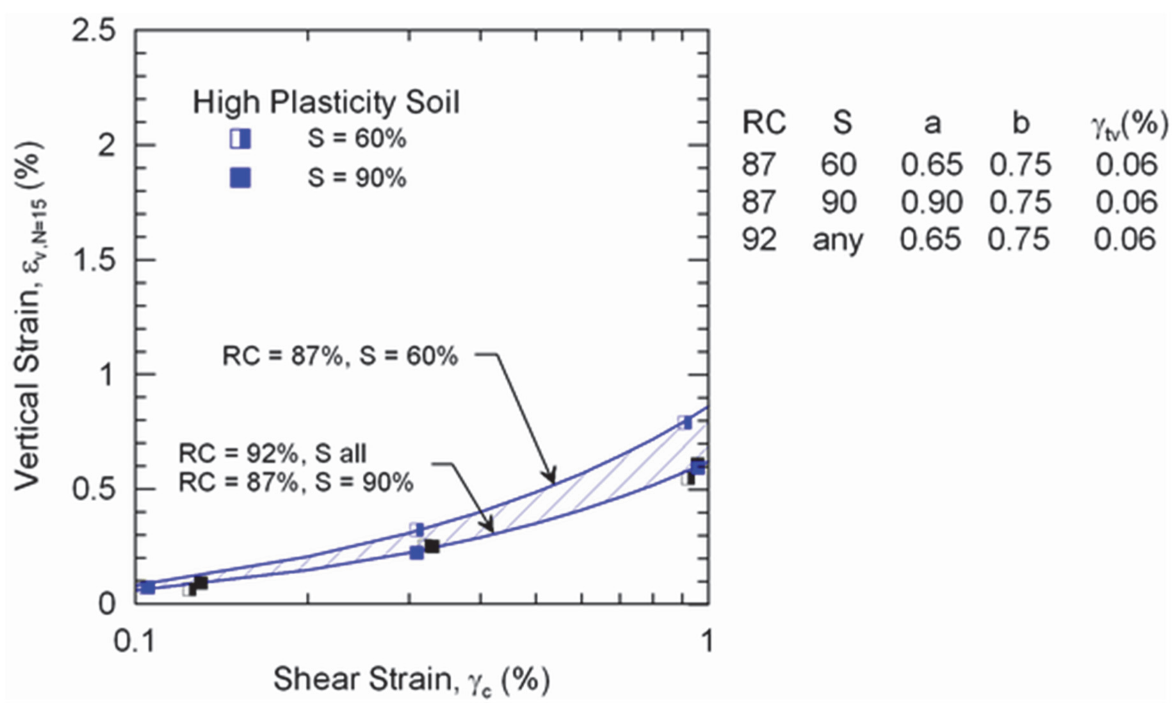


Figure G-12 Volumetric strains for high plasticity fine-grained soil, where PI = 27 and LL = 47 (Stewart et al., 2004b).

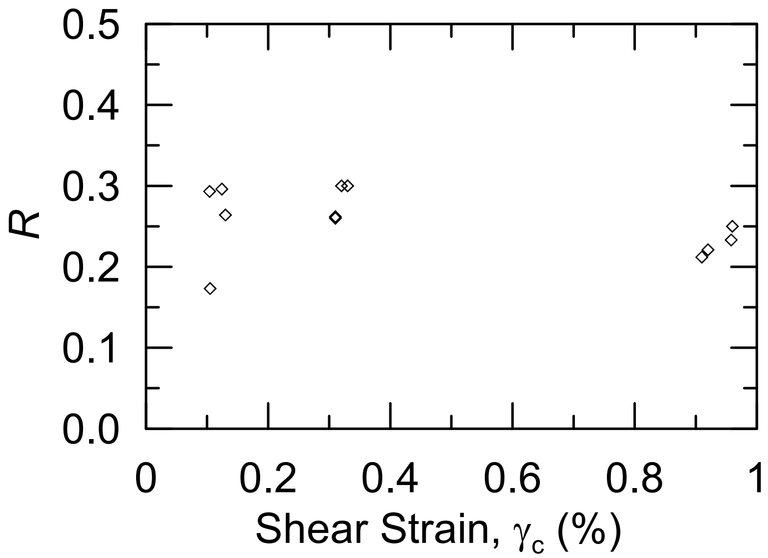


Figure G-13 Values of slope parameter *R* for test results shown in Figure G-12 (Stewart et al., 2004b).

APPENDIX H

Landsliding

Appendix H describes analysis and the associated subsurface explorations and laboratory testing that may be necessary to evaluate earthquake-induced landsliding at a site.

H.1 Subsurface Exploration and Laboratory Testing

The objective of subsurface exploration and laboratory testing is to develop strength and other parameters for use in stability analyses. Subsurface exploration may involve trenching or the drilling of boreholes. Samples are retrieved by hand-carving samples from trenches or downhole-logged boreholes, pushing thin-walled tube samples (e.g., Piston tube, Shelby tube), or driving relatively thick-walled samplers (e.g., Modified California sampler). Guidelines on subsurface exploration and sampling techniques are provided in Chapter 4 and Chapter 6 of Blake et al. (2002). That reference document was prepared for use by practicing engineers and engineering geologists in order to enable slopes in California to be designed in a manner consistent with the intent of the California Seismic Hazards Mapping Act of 1990.

The material samples retrieved during the subsurface exploration program can be used in laboratory testing to estimate shear strength parameters for the slope materials. The estimation of these parameters must be made with due consideration of the drainage conditions during shear, the effects of post-peak reductions of shear strength (i.e., strain softening), and the effects of cyclic degradation, strain rate, anisotropy, and overburden pressure. Practical guidelines for strength parameter estimation that take into consideration each of the above factors is provided in Chapter 7 of Blake et al. (2002), which provides recommendations on the selection of strength parameters for both static and seismic applications.

For seismic applications, the following guidelines for strength parameter selection are offered, many of which are derived from the Blake et al. (2002) reference.

For saturated or nearly saturated soils, undrained strength parameters derived from unconsolidated-undrained (UU) or consolidated-undrained (CU) testing should generally be used. An exception is sliding along pre-existing shear surfaces for which the sheared materials are at residual strength; for such materials, earthquake-induced pore pressures are not expected and drained strength parameters from consolidated-drained (CD) tests can be used. For soils with low levels of saturation (i.e., less than about 90%), drained strength parameters usually provide a conservative estimate of undrained strength parameters for use in design. However, the most accurate strength parameters would still be obtained from rapid, undrained testing of specimens having the same degree of saturation as the in situ materials.

When undrained strengths are used, strength parameters should be selected with due consideration of the effects of strain softening, cyclic degradation, and rate effects. Section 4.9 of Ladd (1991) provides guidance on these issues for static loading of clay. For seismic applications, available test results (Anderson et al., 1988; Azzouz et al., 1989; Zergoun and Vaid, 1994) suggest that for the number of cycles and loading frequencies typical of California earthquakes, the available cyclic shear resistance of PI ≈ 20–40 clays is approximately 80%–100% of the static undrained peak shear strength (lower end of range for large magnitude earthquakes; upper end of range for small magnitude earthquakes).

The selection of strength parameters that are compatible with the expected level of slope deformation is crucial for cemented soils or rock materials. Peak strengths accounting for the effects of cementation can generally be used to evaluate the potential for disrupted slides with the Ashford and Sitar (2002) procedure discussed in Section H.3. However, if any slope deformations were likely to have occurred during the earthquake (i.e., the factor of safety dropped below one at some point during strong shaking), the effects of cementation were likely lost and residual strengths should be used in a displacement-based analysis of slope performance.

With regard to the issue of rate effects, the rapid strain rates applied during seismic loading provide peak dynamic undrained strengths in cohesive soils that are typically 10%–40% larger than peak static undrained strengths measured using typical laboratory testing procedures. However, these effects are largely offset by cyclic degradation effects, and the guidelines presented above (cyclic strengths that are 80%–100% of static undrained shear strengths) are considered appropriate for analyses of seismic stability. Residual strengths are not thought to be significantly influenced by rate effects. Cohesionless soils do not have a significant rate effect, but the potential for soil liquefaction should be investigated.

As with static applications, the interpretation of undrained test results for seismic applications should be performed with due consideration given to the effects of soil anisotropy and overburden pressure.

As described by Blake et al. (2002), methods that can be used to estimate the undrained strengths of soil or rock materials include laboratory triaxial compression or simple shear tests and in situ vane shear tests. Appropriate laboratory testing procedures are described in Section 7.3 of Blake et al. (2002).

H.2 Static Analysis Methods

Static slope stability analyses involve a comparison of the gravity-induced stresses in a slope to the available soil strength and any externally provided resistance (e.g., retaining walls). For slopes in which the shear stresses required to maintain equilibrium under static gravitational loading approach the available shear resistance, the additional dynamic stresses needed to produce instability would be small. Accordingly, the seismic stability of a slope can be closely related to its static stability. For this reason, as well as the close link between many static and seismic stability analysis procedures, static stability analysis procedures are briefly summarized here.

Procedures for the analysis of slope stability under static conditions include limit equilibrium methods and stress-deformation methods. A review of these methods is presented by Duncan (1996). Limit equilibrium methods are used in practice much more frequently than stress-deformation methods, which require the use of finite element or finite difference analyses. Accordingly, the focus of this appendix is on limit equilibrium methods of analysis.

Limit equilibrium methods solve for one or more of the three equations of equilibrium: horizontal force, vertical force, and moment. The equilibrium calculations are performed for a rigid slide mass over a defined slip surface. An assumption inherent to limit equilibrium methods is rigid-perfectly plastic soil behavior, which is depicted in Figure H-1.

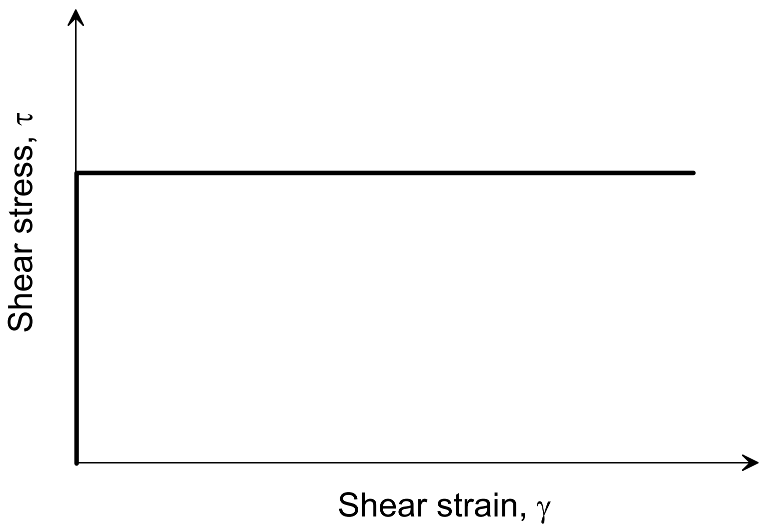


Figure H-1 Stress-strain curve for rigid-perfectly plastic material, which is the assumed condition in limit-equilibrium slope stability analyses (figure credit: J. P. Stewart).

This assumption implies a uniform factor of safety (*FS*) across the slide surface, where *FS* is defined as:

$$FS = \frac{\text{Available Shear Strength}}{\text{Equilibrium Shear Stress}} \tag{H-1}$$

The slope is considered to be at the point of failure when the factor of safety equals one (i.e., the available soil shear strength exactly balances the shear stress induced by gravity). A slope has reserve strength when $FS > 1$. Typical minimum *FS* values for use in slope design are about 1.5 for static long-term stability and 1.25 for static short-term stability.

Generally, the probability of slope failure decreases as the factor of safety increases. However, a unique relationship between probability of failure and *FS* cannot be established because of the wide variability of uncertainties in site-to-site input parameters. In most cases, the largest sources of uncertainty in a slope stability analysis are the soil strength and groundwater conditions. Other factors contributing uncertainty include the imperfect nature of mathematical models for slope stability calculations and the ability of the analyst to find the critical failure surface geometry.

The failure surface that should be analyzed for slope stability must be consistent with the observed slope deformations if such deformations have occurred. In the absence of such field data, any geometric configuration on which the slope might reasonably be envisioned to experience failure should be considered. The intent of analyses is to consider all such surfaces so that the critical surface having the lowest *FS* can be identified. Examples of the types of failure surfaces that should be considered are discussed in Section 9.3 of Blake et al. (2002).

Table H-1 presents a number of commonly used limit-equilibrium methods of slope stability analysis. The various methods of limit equilibrium analysis differ from each other with regard to the equilibrium conditions satisfied and the assumptions made regarding the location and orientation of the internal forces between the assumed slices (which also balances the number of unknowns in the problem with the number of equations). The generalized procedure of slices presented by Morgenstern and Price (1965), Spencer (1967), Sarma (1973), Taylor (1948), and Janbu (1968) satisfy all conditions of equilibrium and involve reasonable assumptions. Bishop's modified method (Bishop, 1955) does not satisfy all conditions of equilibrium but is as accurate as methods that do, provided it is used only for circular surfaces. Duncan (1996) found that these methods provide answers within 5% of each other.

Table H-1 Characteristics of Commonly Used Methods of Limit Equilibrium Analysis

Method	*Equilibrium Conditions Satisfied*	*Shape of Slip Surface*	*Assumptions*
Friction Circle Method (Taylor, 1948)	Moment and force equilibrium	Circular	Resultant tangent to friction circle
Ordinary Method of Slices (Fellenius, 1927)	Moment equilibrium of entire mass	Circular	Normal force on base of slice is $W \cos \alpha$ and shear force is $W \sin \alpha$
Bishop's Modified Method (Bishop, 1955)	Vertical equilibrium and overall moment equilibrium	Circular	Side forces are horizontal
Janbu's (1968) Simplified	Force equilibrium	Any shape	Side forces are horizontal
Lowe and Karafiath's (1960) Method	Vertical and horizontal force equilibrium	Any shape	Side force inclinations are average of slope surface and slip surface (varies from slice to slice)
Janbu's (1968) Generalized Method	All conditions of equilibrium	Any shape	Assumes heights of side forces above the base vary from slice to slice
Spencer's (1967) Method	All conditions of equilibrium	Any shape	Inclinations of side forces are the same for every slice; side force inclination is calculated in the process of the solution
Morgenstern and Price's (1965) Method	All conditions of equilibrium	Any shape	Inclinations of side forces follow a prescribed pattern; side forces can vary from slice to slice
Sarma's (1973) Method	All conditions of equilibrium	Any shape	Magnitudes of vertical side forces follow prescribed patterns

Note: Information in this table is based on Duncan (1996).

H.3 Seismic Analysis Methods

An analysis of seismic slope stability begins with an assessment of whether the earthquake is likely to significantly weaken the slope material, for example through soil liquefaction or through the initiation of deformation in a weakly cemented soil or rock mass that subsequently de-aggregates. If the slope material is potentially susceptible to liquefaction, the engineer must first evaluate whether liquefaction is likely to have been triggered, using the procedures in Appendix F. If liquefaction was likely to have been triggered, appropriate post-liquefaction residual strengths should be used in slope stability analyses. If these strengths are sufficient to maintain static stability (static $FS > 1$), the problem is classified as cyclic mobility and is typically analyzed using displacement-based analysis procedures for a coherent slide mass, or for very flat slopes, the lateral spread analysis methods presented previously. Flow slides occur if the static $FS < 1$ using post-liquefaction strengths. If a flow slide had occurred at a site, it would be obvious from the very large slope displacements that would ensue.

If the problem involves weakly cemented rock or soils, an evaluation of the triggering of deformation can be performed using peak strengths and pseudo-static analysis procedures (Ashford and Sitar, 2002). The intent of the pseudo-static analyses is to check whether the shear stress during earthquake shaking approaches the peak (cemented) strength. If at some point during strong earthquake shaking, the shear stresses match the peak strength, de-aggregation of the material can occur, which will lead to a disrupted slide or fall if the residual strength of the material is less than the static shear stress.

Stability analyses for slopes comprised of materials whose strength is unlikely to be significantly compromised by the earthquake focus on the slope deformations that might accumulate during earthquake shaking. These displacement-based analysis procedures can also be used for cyclic mobility problems in liquefiable soils or the displacements of cemented soils that have become de-aggregated, provided that the residual strength of the material is greater than the static shear stress.

Methods of analysis for disrupted slides and falls and coherent slides are presented in the following sections.

H.3.1 Analysis of Weakly Cemented Soil or Rock Slopes

Ashford and Sitar (2002) recommend the use of a pseudo-static approach for the analysis of landslide potential in steep, weakly cemented slopes. Pseudo-static methods of seismic slope stability analysis involve the use of a destabilizing horizontal seismic coefficient (k) within a conventional limit equilibrium slope stability calculation. The seismic coefficient represents the fraction of the weight of the sliding mass that is applied as an equivalent horizontal force acting through the centroid of the slide mass. The factor of safety against shear failure is evaluated with the equivalent horizontal force applied to the slope.

The slope geometry utilized in the development of the Ashford and Sitar (2002) procedure is shown in Figure H-2.

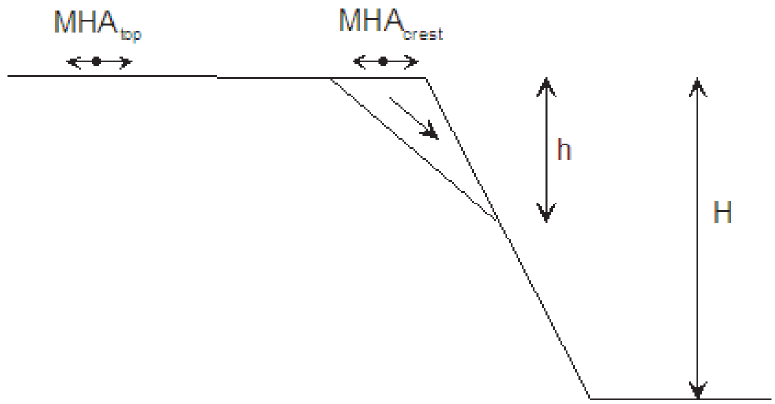


Figure H-2 Schematic illustration of slope geometry considered by Ashford and Sitar (2002) for steep slopes of weakly cemented rock or soil (figure credit: J.P. Stewart).

The seismic coefficient is evaluated as follows:

1. Evaluate the maximum horizontal acceleration in the "free field" behind the slope crest (MHA_{top}). In this context, free field refers to motions not influenced by surface topography. If the site condition behind the slope crest is not a standard reference site condition (i.e., rock or soil), the use of site amplification factors (Stewart et al., 2003) may be appropriate during the estimation of MHA_{top}.
2. Evaluate the maximum horizontal acceleration at the slope crest as $MHA_{crest} = 1.5 \times MHA_{top}$, to account for topographic amplification effects.
3. Estimate slope height (H) and distance from slope crest to base of slide plane, h.
4. Estimate the maximum seismic coefficient likely to occur within the slope ($k_{\max}$) using Figure H-3. Ashford and Sitar (2002) indicate that the upper end of the range of $k_{\max}/(MHA_{crest}/g)$ values should be used for steep slopes (around 75 degrees), whereas the average of the Makdisi and Seed (1978) range is appropriate for less steep slopes (45 degrees).
5. The horizontal seismic coefficient is taken as $0.65\times k_{\max}$.
6. A pseudo-static stability analysis is performed using peak strengths for the slope material and the seismic coefficient of $0.65\times k_{\max}$. According to Ashford and Sitar (2002), slopes with $FS > 1$ are unlikely to have significantly displaced during earthquake shaking. Slopes with $FS < 1$ may have had some displacement and were likely in danger of de-aggregating. The stability of the de-aggregated slide mass should be evaluated using the displacement-based analyses described in the following section. Such analyses should be performed using residual strength parameters.

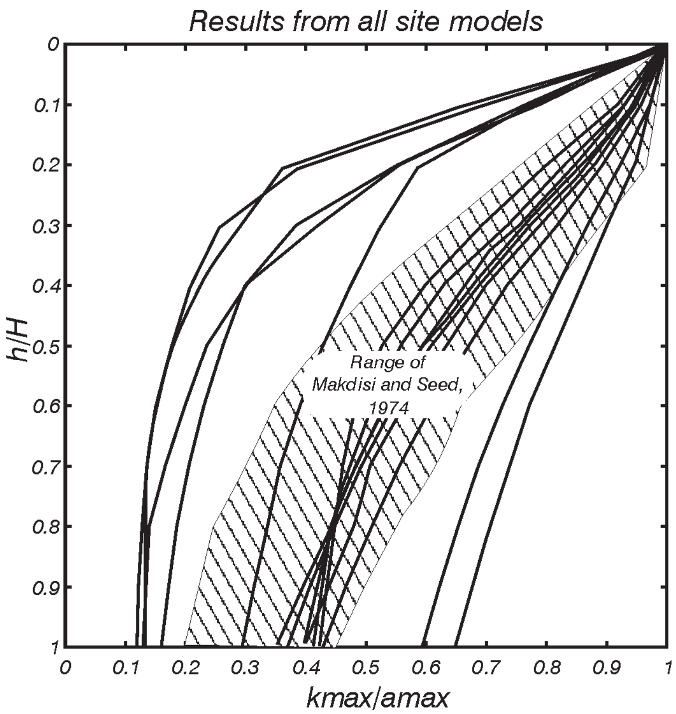


Figure H-3 Seismic coefficient profiles evaluated for steep slopes compared to range of values from Makdisi and Seed (1978) (Ashford and Sitar, 2002).

H.3.2 Displacement-Based Analysis Methods

The seismic performance of a slide mass can be evaluated using an analysis procedure that accounts for the time-varying nature of the seismic excitation of the mass. Newmark (1965) developed such a procedure by recognizing that displacements accrue in a slope as a result of increments of time during which the seismic excitation causes the factor of safety to drop below one. As illustrated in Figure H-4, Newmark drew an analogy between this situation and that of a rigid block resting on an inclined plane, which will slide down the plane whenever the inertial excitation produces basal stresses that exceed the shear strength at the block-plane interface.

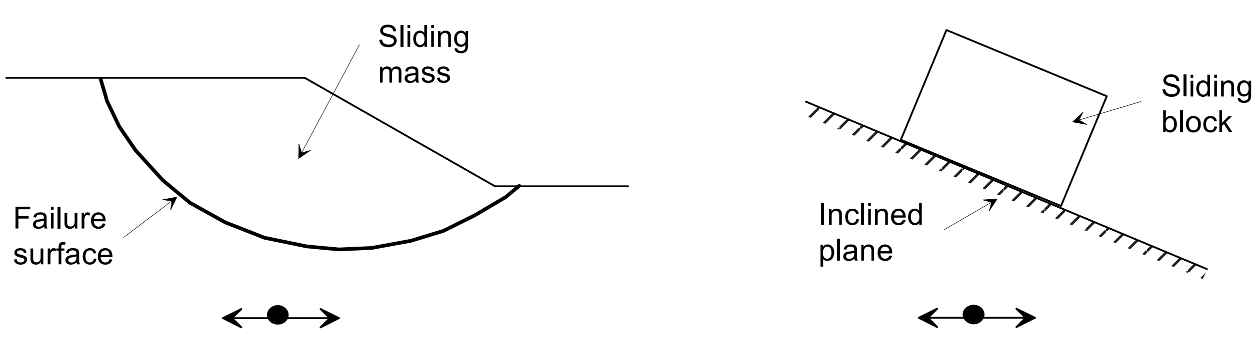


Figure H-4 Analogy between a potential earthquake-induced landslide (left) and a rigid block resting on inclined plane (right) (figure credit: J. P. Stewart).

Using Newmark's model, the displacement of a rigid block can be calculated for any base excitation time history if the acceleration that causes the initiation of slip is known. This acceleration is known as the yield acceleration and is denoted a_y. There is a corresponding seismic coefficient that is referred to as $k_y = a_y/g$, where g = acceleration of gravity. Parameter k_y can be calculated in conventional limit equilibrium stability calculations by introducing static lateral forces of $k \times W$ (where W = weight of slide mass) through the centroid until the value of k that reduces the factor of safety to one is identified. This value of k is equal to k_y. Considerations associated with the selection of strength parameters for use in this evaluation of k_y were presented earlier in this appendix.

As illustrated in Figure H-5, the calculation of displacement given an accelerogram and a_y involves first integrating across the portion of the accelerogram where the block and the base will have differing velocities. As shown in the Figure H-5, the differential velocity begins at the instant of time when acceleration first exceeds a_y (Point A) and increases throughout the time period during which $a > a_y$. When the acceleration drops below a_y (Point B), the differential velocity is at a local maximum. Differential velocity will decrease while $a < a_y$ until it goes to zero (Point C), at which time the block and base will again resume coherent motion until the next occurrence of basal slip. Once the time history of differential velocity has been computed as described above (and as represented in the middle frame of Figure H-5), the differential displacement is simply calculated by integrating across the differential velocity time history (as shown in the bottom frame of Figure H-5).

The above procedure is convenient to apply, especially with the availability of modern computer programs that can efficiently perform calculations for many time histories (Jibson and Jibson, 2002). However, a number of issues can critically affect the outcome of such analyses and should be borne in mind by the geotechnical consultant, such as:

The slide mass above a basal slip plane is not truly rigid, and the dynamic response of the mass could give rise to: (a) amplification or de-amplification of the base motion depending on the velocity structure of the site and the potential for resonance between the input motion and slide mass; and (b) wave reversals within the slide mass depending on the frequency content of incident waves and depth and velocity structure of the slide mass. These effects will be collectively referred to as vertical ground motion incoherence and have been investigated by a number of researchers, including Kramer and Smith (1997) and Bray and Rathje (1998).

Calculated displacements are highly sensitive to characteristics of the input motions, such as amplitude, duration, and frequency content. Moreover, even for a set of time histories for which these characteristics are consistent, calculated displacements can show significant variability due to essentially random phasing of the waveforms. Accordingly, time histories must be carefully selected to match the magnitude and site-source distance associated with the causative earthquake, and a sufficient number of time histories should be selected to enable both the median displacement and the dispersion of displacements to be reliably characterized.

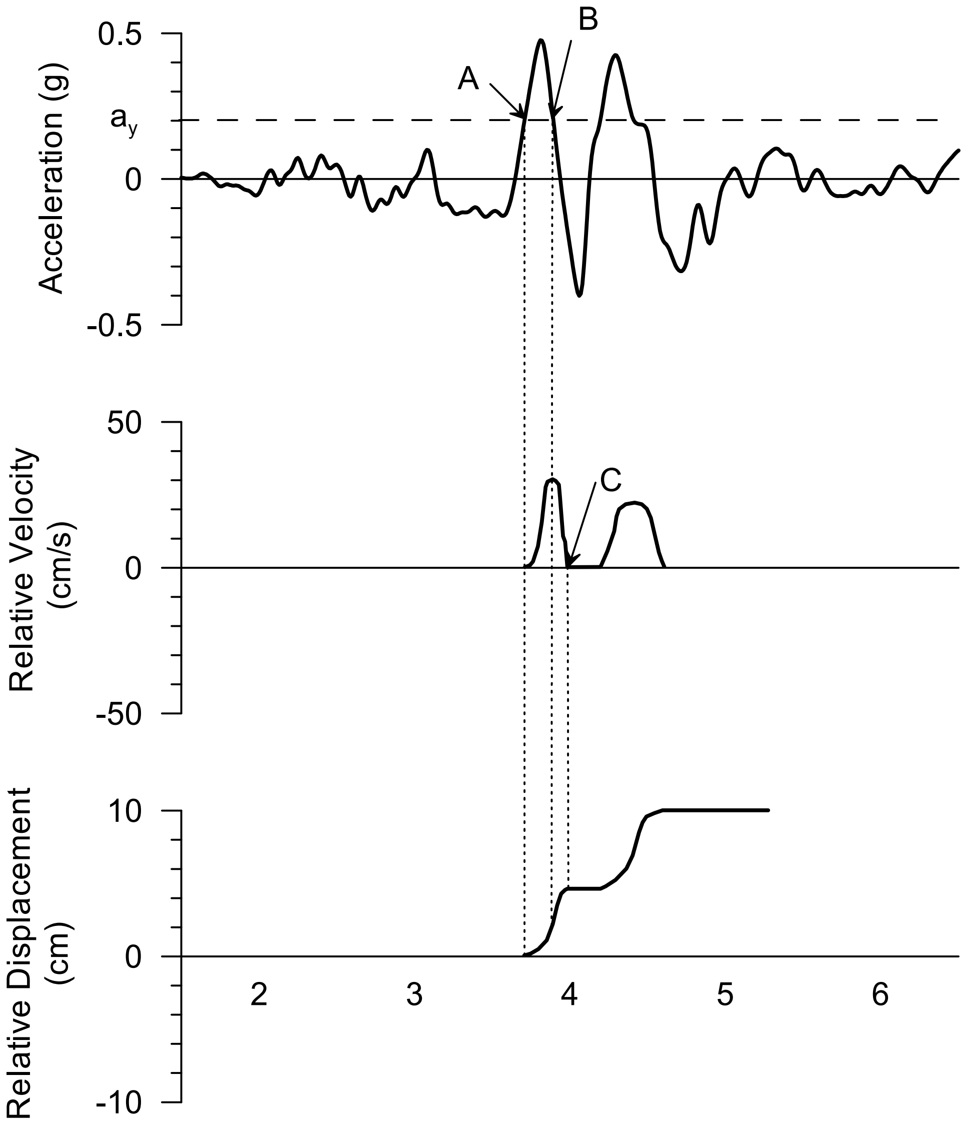


Figure H-5 Development of permanent slope displacement for earthquake ground motion (adapted from: Wilson and Keefer, 1985).

The shear strength parameters used to evaluate yield coefficient (k_y) must be appropriate for the seismic loading condition. These parameters will typically be different from those used for static stability analyses, as discussed above.

The occurrence of basal slip of a slide mass causes its motions to deviate from those that would be present in the absence of slip. When analysis of the dynamic response of the slide mass is performed independently of the analysis of relative displacement, the analyses are said to be de-coupled. A coupled analysis considers the dynamic response and the basal slip together. Displacements calculated from de-coupled and coupled analyses generally differ (Lin and Whitman, 1983; Gazetas and Uddin, 1994; Kramer and Smith, 1997; Rathje and Bray, 2000).

The implication of the vertical ground motion incoherence effects discussed above is that acceleration time histories selected from a strong motion database should not be used in their as-recorded state for Newmark sliding block analyses if the dynamic response of the slide mass is likely to be significant. The slide mass response is insignificant if the wavelength of the incident waves significantly exceeds the slide depth, or expressed another way, the period of the slide mass (T_s) is much smaller than the mean period of the input motion (T_m, evaluated from Rathje et al., 1998).

Bray and Rathje (1998) recommend that if $T_s/T_m < 0.2$, the slide mass response is insignificant, and the mass can be considered to be rigid. However, for $T_s/T_m > 0.2$, a ground response analysis should be performed that is appropriate for the site geometry to evaluate the horizontal equivalent acceleration time history, HEA(t). HEA/g represents the ratio of the time-dependent horizontal inertial force applied to a slide mass during an earthquake to the weight of the mass. The maximum value of HEA is denoted MHEA, which can be related the maximum seismic coefficient by $k_{\max} = \text{MHEA}/g$. HEA time histories can generally be evaluated from one- or two-dimensional ground response analyses using computer programs, such as SHAKE or QUAD4M (Idriss and Sun, 1991; Hudson et al., 1994). Rathje and Bray (1999) have found that 1-D analyses generally provide a conservative estimate of HEA(t) for deep sliding surfaces within two-dimensional slope geometries and a slightly unconservative estimate for shallow surfaces near slope crests.

The implication of the difference between sliding block displacements calculated from de-coupled and coupled analyses is that the more conventional, de-coupled analyses can produce biased estimates of slope displacement. Rathje and Bray (2000) found that de-coupled analyses are significantly conservative (over-predict displacements) for $T_s/T_m < 1.0$. For larger period ratios, de-coupled displacements may be conservative or unconservative, the unconservative situation being more likely for $k_y/k_{\max} > 0.4$. As of this writing, there are no widely distributed computer programs available for the analysis of coupled sliding block displacements.

As an alternative to the relatively complex Newmark integration analyses discussed above, a number of simplified procedures have been developed that can be used to estimate Newmark sliding block displacements. These procedures have been developed by a number of investigators, although perhaps the most widely accepted procedures are those of Bray and Travasarou (2007), Bray and Rathje (1998), and Bray et al. (1998). Bray and Rathje (1998) was originally developed for landfills but has also found widespread use for hillside residential and commercial construction (Blake et al., 2002). Bray and Travasarou (2007) was motivated by the inherent limitations of decoupled analysis and the significant increase in quality ground motion records that were available following several large earthquakes in the late 1990s and early 2000s. Recent methods made available in the literature have worked toward a probabilistic approach (Bray et al., 2018; Rathje and Saygili, 2011).

Bray and Rathje (1998) is described in the following paragraphs, to illustrate the general procedure for a displacement-based analysis and several concepts of importance in applying the simplified procedures.

The procedure has two basic steps: (1) analysis of the seismic demand accounting for vertical incoherence effects; and (2) evaluation of normalized displacement.

Bray and Rathje (1998) define the spatially averaged peak acceleration of a slide mass as the maximum horizontal equivalent acceleration (MHEA). Bray and Rathje evaluated MHEA as a function of MHA_r from calculations of wave propagation through equivalent one-dimensional slide masses. The results of these calculations are shown in Figure H-6, where MHEA is normalized by the product of MHA_r and a nonlinear response factor (NRF). Parameter NRF accounts for nonlinear ground response effects as vertically propagating shear waves pass through the slide mass. Parameter MHA_r is used as the normalizing ground motion even for sites where the foundation materials are soil because site condition was not found to significantly affect MHEA (except for deep soft clay sites such as NEHRP E sites, for which site-specific analyses were recommended). The ratio $MHEA/(MHA_r \times NRF)$ differs from one as a result of vertical ground motion incoherence within the slide mass and is related in Figure H-6 to the ratio of the small-strain period of the sliding mass (T_s) to the mean period of the input motion. The ratio $MHEA/(MHA_r*NRF)$ is less than one for $T_s/T_m > \sim 0.5$ and is variable with an average of about 1.0 for $T_s/T_m < \sim 0.5$.

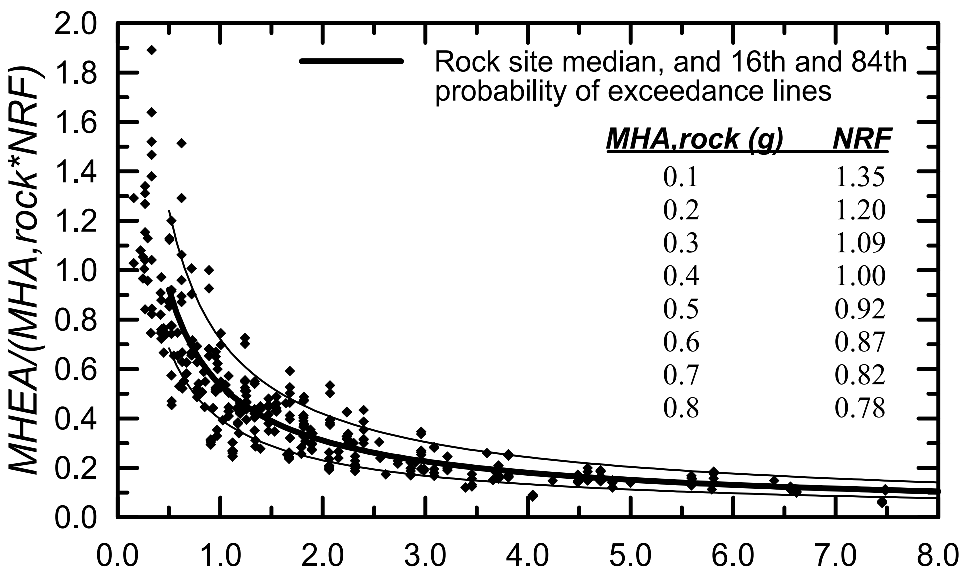


Figure H-6 Normalized MHEA for deep-seated slide surface vs. normalized fundamental period of slide mass (adapted from: Bray et al., 1998).

Bray and Rathje (1998) developed a statistical model that relates slope displacements from a Newmark-type analysis (u) to the amplitude of shaking in the slide mass ($k_{\max} = MHEA/g$), significant duration of shaking (measured as the time between 5% to 95% normalized Arias intensity, $D_{5\text{-}95}$) and the ratio $k_y/k_{\max}$. A statistical model was established from regression analysis of 309 Newmark-displacement values calculated from ground motion records from earthquakes of magnitude 6.25 to 8 at each of four $k_y/k_{\max}$ ratios. The model and data are shown in Figure H-7 and indicate a lognormal distribution of normalized displacement $u/(k_{\max}D_{5\text{-}95})$ for a given $k_y/k_{\max}$ ratio.

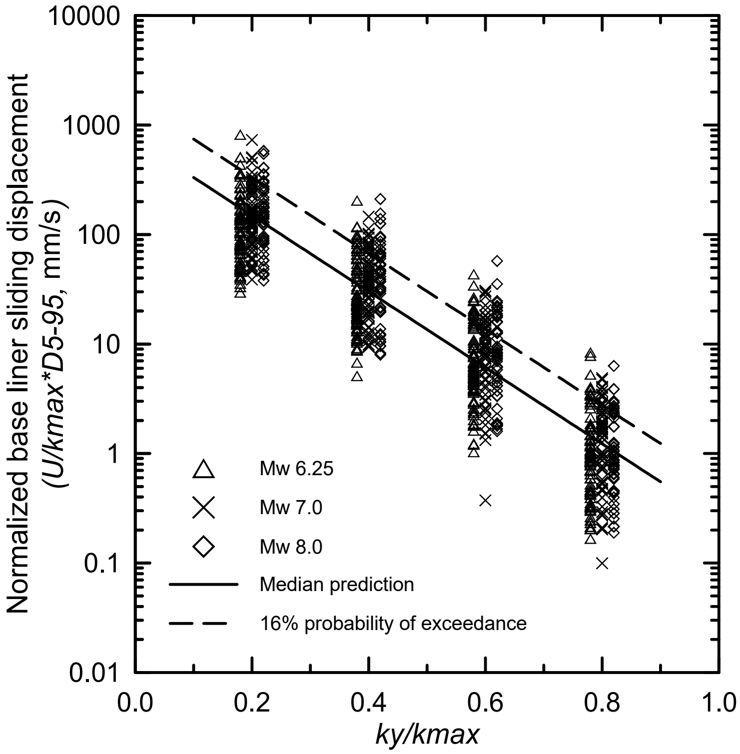


Figure H-7 Normalized sliding displacement (adapted from: Bray and Rathje, 1998).

The median of this lognormal distribution is described by,

$$\log_{10}\left(\frac{u}{k_{\max}\times D_{5-95}}\right)=1.87-3.477\frac{k_y}{k_{\max}} \tag{H-2}$$

where u is the median displacement in cm. The standard deviation is 0.35 in $\log_{10}$ units.

Whether evaluated through formal Newmark integration or the simplified procedure described above, the calculated displacement u should be recognized as an index of slope performance and does not necessarily correspond to the actual displacement of the slope. Nonetheless, the calculated displacement can be used to aide in the evaluation of whether earthquake induced landslide movements were likely to have occurred at the site. If the calculated displacements are zero with reasonable assumptions of ground motion and soil strength characteristics, then earthquake-induced slope displacements were unlikely to have occurred. Likewise, nonzero calculated displacements suggest movements were possible, especially if corroborated by field observations of distress where the landslide slip surface intersects the ground surface.

For cases where earthquake-induced landslide movements were likely to have occurred, the calculated Newmark displacement can be used along with suitable field observations to evaluate the likely effect of landslide movements on surface improvements. In general, the larger the calculated Newmark displacement, the more likely earthquake-induced landslide movements were to have damaged surface improvements. Existing guidelines (Blake et al., 2002) suggest that calculated Newmark displacements less than 2 inches (5 cm) in occupied structures and less than 6 inches (15 cm) outside of occupied structures are generally acceptable for design purposes. However, displacements smaller than those threshold values could potentially cause damage, depending on the degree of localization of the ground displacements and the structural integrity of the affected improvements. The best way to evaluate the impact of landslide movements on improvements is by direct inspection of the improvements in the field, including measurements of crack widths, an assessment of the freshness of any cracks and floor elevation surveys.

APPENDIX I

Retaining Wall Deformation

Appendix I describes analysis and the associated subsurface explorations and laboratory testing that may be necessary to evaluate earthquake-induced retaining wall deformations at a site.

I.1 Subsurface Exploration and Laboratory Testing

The objectives of subsurface exploration and laboratory testing for retaining wall studies are to determine the type and distribution of foundation and backfill materials, to evaluate soil strength and index properties, and to inspect the drainage system for signs of damage.

Soil samples may be obtained from test pits or from drilling of boreholes. A sufficient number of exploration points (e.g., test pits or borings) should be used to reasonably evaluate variations in soil properties along the wall. Generally, this will require subsurface exploration at a minimum of two locations; additional exploration points may be warranted depending on the extent of the wall and backfill material.

Soil sample testing may include in situ densities evaluated with in situ sand cone tests (ASTM D 2419) performed within test pits or through laboratory testing of samples retrieved in the field (ASTM D 2937). Samples to be used for such purposes should be disturbed to the least extent possible, as disturbance will change sample density, as discussed in Appendix G. Additional laboratory testing helpful for the retaining wall analyses include the water content (ASTM D 2937), gradation (ASTM D 422 or ASTM D 1140), liquid limit and plastic limit (ASTM D 4318).

The material samples retrieved during the subsurface exploration program can be used in laboratory testing to estimate shear strength parameters for the backfill materials and materials in front of the retaining wall that may develop passive earth pressures. The estimation of these parameters is discussed in Appendix F.

Examination of the drainage system and cantilever retaining wall stem may be performed within a test pit excavated behind the face of the wall at the location of maximum wall rotation.

I.2 Static Analysis Methods

The resultant of active earth pressures from the backfill typically exceeds the resultant from passive earth pressures (if present) below the toe of the wall. Stable equilibrium of retaining walls is achieved by earth pressures mobilized along the base of the retaining structure. These long-term static stresses control the static stability of the wall and strongly influence the seismic stability as well. Even under static conditions, the stresses acting on a retaining wall are highly indeterminate. Stress-deformation methods, which require the use of finite element or finite difference analyses, are rarely used in practice to analyze retaining walls. Limit equilibrium analyses are common in practice for retaining structures and utilize simplifying assumptions to reduce the indeterminacy to three equations of equilibrium: horizontal force, vertical force, and moment. For these analyses, a failure state is assumed to exist along defined slip surfaces within the backfill behind the wall (active state) and in foundation materials below the wall toe (passive state). The shear stresses along those surfaces are assumed to be the shear strength of the material. As with limit equilibrium slope stability analyses, the constitutive assumption along the slip surface is rigid-perfectly plastic soil behavior, as shown in Figure H-1.

The focus of the remainder of this section is on limit equilibrium methods of analysis. A summary of limit equilibrium methods for planar and curved failure surfaces may be found in most geotechnical engineering textbooks (for example, see ASCE, 1994a). For retaining walls, limit equilibrium enforces basic stability requirements for horizontal forces (sliding), vertical forces (bearing capacity), and moment (overturning) with a factor of safety (*FS*), where *FS* is defined as:

$$FS = \frac{\text{Available Resisting Forces or Moments}}{\text{Driving Forces or Moments}} \tag{I-1}$$

The retaining wall is considered to be at the point of failure when the factor of safety equals one, (i.e., the wall has moved sufficiently to fully mobilize the shear strength of the soil on the slip surface). A retaining wall has reserve capacity when $FS > 1$. Table I-1 contains a summary of the typical factors of safety utilized for retaining wall design.

Table I-1 Summary of the Typical Factors of Safety Utilized for Retaining Wall Design [(1)]

	Factor of Safety (or Criteria)		
Loading Condition	*Usual*	*Unusual*	*Earthquake*
Base sliding	1.5	1.33	1.1
Bearing capacity	3	2	> 1
Overturning criteria: minimum base area in compression (soil foundation)	100% [(2)]	75% [(2)]	Resultant within base
Overturning criteria: minimum base area in compression (rock foundation)	75% [(2)]	50% [(2)]	Resultant within base

[(1)] Information in table based on ASCE (1994a).

[(2)] Less base area in compression than the minimum shown may be acceptable provided that adequate safety against unacceptable differential settlement and bearing failure is obtained.

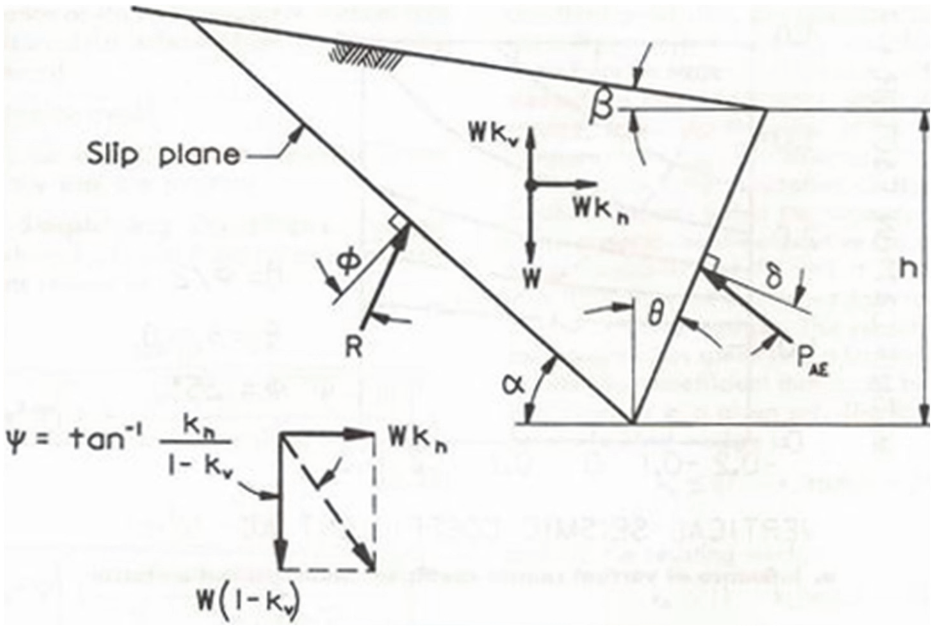
I.3 Seismic Analysis Methods

A common approach to analysis of retaining walls for design is extending static limit equilibrium analysis to pseudo-static conditions and checking if equilibrium is satisfied (Okabe, 1926; Mononobe and Matsuo, 1929). Collectively, these early methods are known as the Mononobe-Okabe methods, which are summarized in Anderson et al. (2009). The evaluation of the triggering of deformations can be performed using pseudo-static analysis procedures; however, no information regarding the magnitude of wall displacements is obtained. Displacement-based analysis of cantilever and gravity retaining walls may be performed in a manner analogous to the Newmark sliding block procedure discussed previously (Richards and Elms, 1979; Whitman and Liao, 1985; and Elms and Richards, 1990). For mechanically stabilized earth (MSE) slopes and walls, a deformation-based analysis procedure has been proposed by Nova-Roessig (1999).

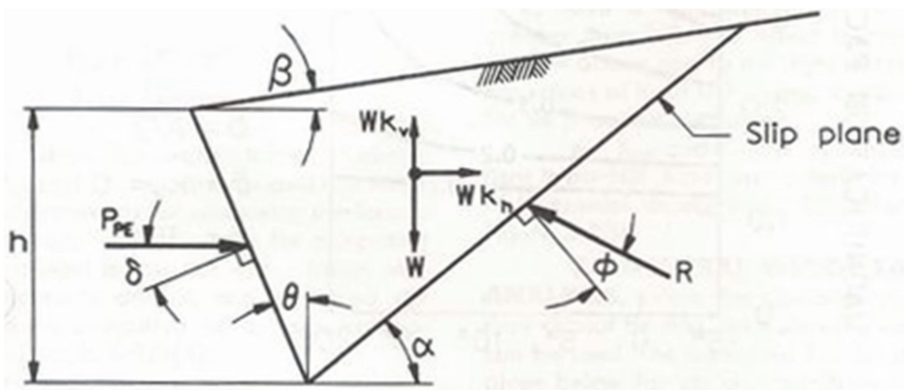
I.3.1 Pseudo-Static Analysis: Yielding Walls

If cantilever and gravity retaining walls displace sufficiently to fully mobilize the shear strength of the soil on a slip surface (i.e., yielding walls), displacements can be estimated by using pseudo-static analysis. Okabe (1926) and Mononobe and Matsuo (1929) developed a method for analyzing seismic earth pressures on retaining structures that has become known as the Mononobe-Okabe (M-O) method. The method is an extension of the Coulomb sliding wedge theory, taking into account horizontal and vertical inertial forces acting on the soil and assuming homogeneous cohesionless backfill material. The Mononobe-Okabe analysis is described in more detail in Seed and Whitman (1970), Whitman and Liao (1985), and ASCE (1994a).

Figure I-1 shows free body diagrams of forces acting on a driving (active) wedge and resisting (passive) wedge subject to Mononobe-Okabe loading assumptions for unsaturated soil conditions.



(a) Mononobe-Okage (active) wedge



(b) Passive wedge

Figure I-1 Forces acting on a Mononobe-Okabe active and passive soil wedge (ASCE, 1994a).

The total driving (active) and resisting (passive) forces on a wall are expressed as:

$$P_{AE} = P_A + \Delta P_{AE} = \frac{1}{2} K_{AE} \gamma H^2 (1 - k_v) \tag{I-2}$$

$$P_{PE} = P_P + \Delta P_{PE} = \frac{1}{2} K_{PE} \gamma H^2 (1 - k_v) \tag{I-3}$$

where:

$$K_{AE} = \frac{\cos^2(\varphi - \Psi - \theta)}{\cos\Psi \cos^2\theta \cos(\Psi - \theta + \delta)\left[1 - \sqrt{\dfrac{\sin(\varphi+\delta)\sin(\varphi - \Psi + \beta)}{\cos(\beta - \theta)\cos(\Psi - \theta + \delta)}}\right]^2} \qquad \text{(I-4)}$$

$$K_{PE} = \frac{\cos^2(\varphi - \Psi + \theta)}{\cos\Psi \cos^2\theta \cos(\Psi - \theta + \delta)\left[1 - \sqrt{\dfrac{\sin(\varphi+\delta)\sin(\varphi - \Psi + \beta)}{\cos(\beta - \theta)\cos(\Psi - \theta + \delta)}}\right]^2} \qquad \text{(I-5)}$$

where:

P_A, P_P	= static active and passive forces, respectively
ΔP_{AE}, ΔP_{PE}	= dynamic active and passive forces, respectively
P_{AE}, P_{PE}	= sum of static and dynamic active and passive forces, respectively
γ	= unit weight of soil
ϕ	= friction angle of soil
k_v	= vertical seismic coefficient
k_h	= horizontal peak seismic coefficient
Ψ	= $\tan^{-1}\left(\dfrac{k_h}{1-k_v}\right)$
H	= height of wall
θ	= inclination of wall with respect to vertical
δ	= wall friction angle
β	= inclination of backfill behind wall

The quantity k_h in the above equations represents the effective peak horizontal seismic coefficient for the backfill materials, which is equivalent to the effective horizontal acceleration normalized by g. (The quantity k_h as used here for retaining walls is analogous to the peak seismic coefficient used for landslide studies, $k_{\max}$. See Appendix H for additional discussion on this issue for the landslide problem.) Accordingly, k_h would in general be related to the peak acceleration at the base of the wall, ground motion amplification that may occur across the height of the wall, and wave reversal and resonance effects that may occur across the height of the wall. The latter two effects also impact the location of the resultant dynamic thrust force. In the limiting case of rigid backfill soil where no amplification or wave reversal effects are possible, the acceleration acting on the backfill would be uniform with height and would match the base acceleration (i.e., $k_h = PHA/g$, where PHA is the peak acceleration at the ground surface for level ground conditions). Since the active wedge is roughly triangular in shape, the resulting inertial thrust would act at a point that is one-third of the wall height down from the surface of the backfill (i.e. h_d =

0.33H). Use of $h_d = 0.33H$ and $k_h = PHA/g$ to characterize the dynamic component of Equation I-2 were recommended by Seed and Whitman (1970) for walls that are allowed to displace relative to the backfill.

Steedman and Zeng (1990) have investigated the dynamic amplification of ground motion across the height of retaining walls with uniform backfill and foundation soils. Thus, their analyses consider the effects of wave reversal and resonance effects but not amplification due to impedance contrasts. Figure I-2 shows the location of the dynamic resultant thrust force as a function of wall height normalized by wavelength $\lambda = V_s/f$ (where V_s = shear wave velocity of backfill and f = frequency of wave). Note that for very long wavelengths ($H/\lambda \rightarrow 0$), the distance from the top of the backfill to the dynamic resultant (h_d) approaches 0.33H, as noted above. This condition persists to $H/\lambda \approx 1/8$, which corresponds to $f = V_s/8H$. Accordingly, wave reversal and resonance effects are negligible for components of ground motion with frequencies less than half the fundamental frequency of the unrestrained backfill (i.e., $f < V_s/8H$), which encompasses the range of frequencies containing most of the seismic energy for many practical problems.

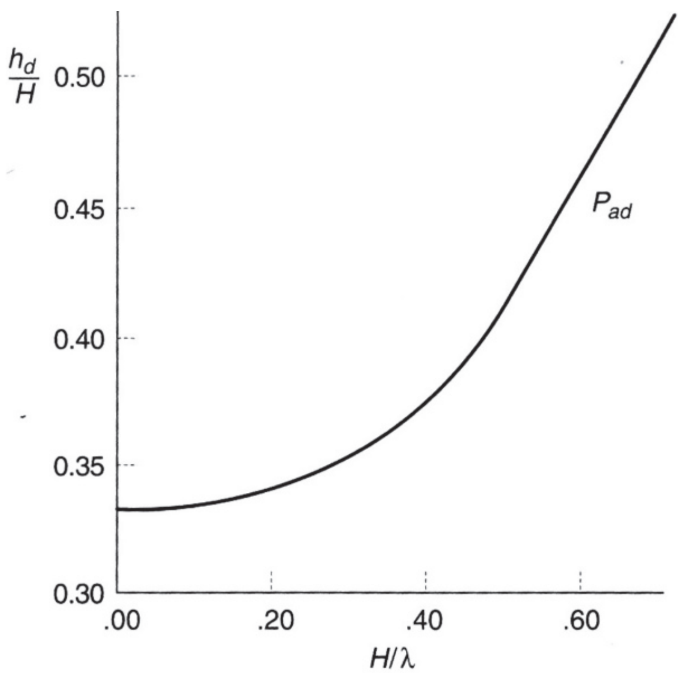


Figure I-2 Location of dynamic thrust at the instant of maximum overturning moment for $k_h = 0.2$ (Steedmand and Zeng, 1990)

For tall walls where $f > V_s/8H$, Steedman and Zeng's results in Figure I-2 show that the location of the resultant moves down the wall and can approach mid-height ($h_d \approx 0.5H$). As this occurs, the distribution of dynamic pressures becomes non-triangular, as shown for example in Figure I-3, where z is the depth from the top of wall.

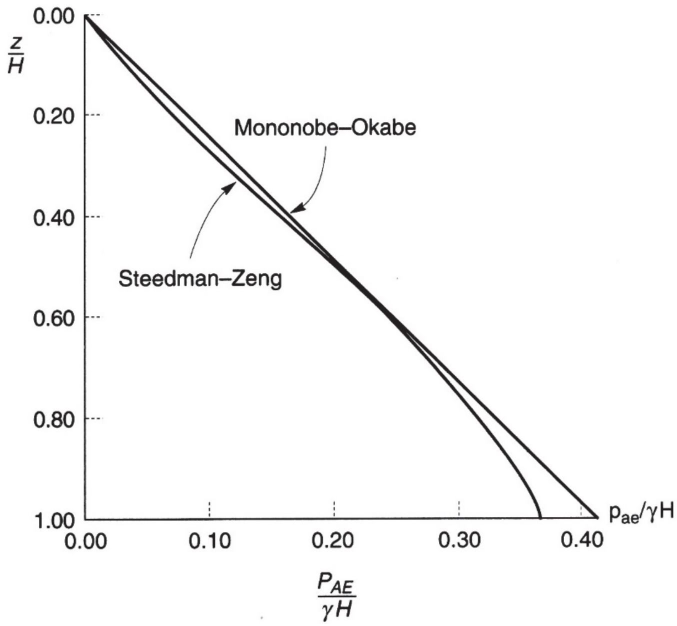


Figure I-3 Comparison of normalized dynamic pressure distributions for the Steedman-Zeng and the Mononobe-Okabe methods assuming k_h = 0.2 and h/λ = 0.3 (Steedman and Zeng, 1990).

Based on the above considerations, the pseudo-static analysis procedure for a post-earthquake analysis is as follows:

1. Evaluate the peak horizontal acceleration (*PHA*) and peak vertical acceleration (*PVA*) in the "free field" behind the retaining wall. In this context, free field refers to motions not influenced by surface topography or other structures. Nonplanar top surfaces and any surcharge loading need to be included in these analyses. In addition, the effects of significant impedance contrasts in the backfill materials should be accounted for in the evaluation of *PHA* and *PVA*.

2. Evaluate the effective peak horizontal seismic coefficient (k_h) acting on the backfill in consideration of potential wave reversal and resonance effects. In most practical situations, it is expected that the wall height is sufficiently small that the backfill can be considered to be rigid, in which case k_h = *PHA*/*g*. Similarly, k_v in most cases can be taken as *PVA*/*g*.

3. Estimate the static and dynamic components of active and passive earth pressures including inertial forces using Equation I-2 through Equation I-5.

4. Calculate the *FS*s for retaining wall stability.

Retaining walls with *FS* > 1 are likely to have relatively small displacements associated with the mobilization of the soil shear strength. Slopes with *FS* < 1 may have had larger displacements. Retaining wall displacements can be estimated using analysis procedures described in the following section.

The above procedure is straightforward to apply; however, several issues should be considered when interpreting the results, such as:

- For cantilever walls, bending stresses must be checked at the base of the stem to determine if over-stressing has occurred.
- For yielding walls, the analysis is subject to all of the same assumptions and limitations as the Coulomb analysis, and the results should be interpreted accordingly. Specifically, the wall is assumed to be free to move sufficiently to enable full soil strength to be mobilized in the backfill and in the foundation soils below the toe of the wall.
- For MSE walls, additional seismic analysis needs to be performed to include checks on internal stability, such as a reinforcement pullout, reinforcement rupture, or separations of the reinforcements from the facing components. Volumetric strains (seismic compression) of the soil within the reinforced zone should also be addressed (see Appendix G).

Recent studies have discussed problems inherent to the use of M-O methods that are often overlooked in engineering practice. These include (Anderson et al., 2009):

- Cohesive backfill material for which both cohesion and friction angle strength parameters dominate soil behavior, or for inhomogeonous backfill.
- Sloping backfill, which may lead to unrealistically large dynamic active earth pressures.
- Scenarios where high values of k_h or k_v can lead to an infinitely large dynamic earth pressure.
- Experimental work based on geotechnical centrifuge testing and dynamic numerical solutions have shown the M-O methods to be either overly conservative (Al Atik and Sitar, 2010) or unconservative (Ostadan, 2005). This issue can be explained through a kinematic soil-structure interaction framework focusing on the ratio of: (1) wavelength of vertically propagating shear waves; and (2) wall height (Brandenberg et al., 2015), an improvement upon the widely used M-O framework. Despite these limitations, the M-O method is commonplace in practice and has utility, with the aforementioned caveats, for use in assessing earthquake-induced wall movements.

I.3.2 Pseudo-Static Analysis: Non-Yielding Walls

If the retaining wall does not yield sufficiently to fully mobilize the shear strength of the soil, then neither active nor passive earth pressures can develop. Examples include basement walls, retaining walls adjacent to structures that restrain the walls at the top and bottom, and large gravity walls over rough bases. Wood (1973) studied the response of seismic pressures on smooth, rigid, nonyielding walls with homogeneous linear elastic backfills.

In the absence of dynamic amplification across the height of the backfill (i.e., $f < V_s/8H$), the dynamic thrust and overturning moments can be expressed as follows:

$$\Delta P_{eq} = \gamma H^2 \frac{a_h}{g} F_P \tag{I-6}$$

$$\Delta M_{eq} = \gamma H^3 \frac{a_h}{g} F_M \tag{I-6}$$

where:

ΔP_{eq}, ΔM_{eq}	= dynamic thrust and overturning moment, respectively, on rigid wall
a_h	= amplitude of harmonic base acceleration
g	= acceleration of gravity
F_P, F_M	= dimensionless dynamic thrust and moment factors
γ	= unit weight of soil
H	= height of wall

The dimensionless dynamic thrust and moment factors are shown in Figure I-4 and Figure I-5, respectively, where L is length as defined Wood (1973). Wood recommends that the dynamic thrust be placed at a height of

$$h_{eq} = \frac{\Delta M_{eq}}{\Delta P_{eq}} \tag{I-7}$$

above the bottom of the wall (where $h_d = H - h_{eq}$). This corresponds to a value of $h_d = 0.37H$ for many cases.

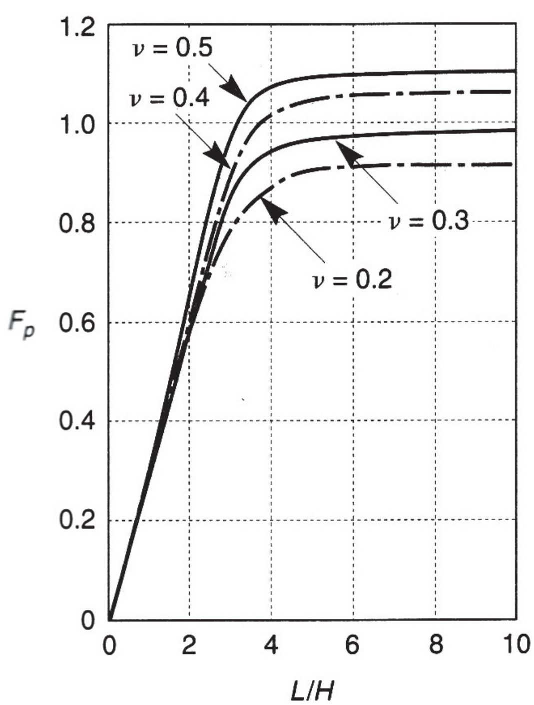


Figure I-4 Dimensionless thrust factor for various geometries and soil Poisson's ratio values, ν (Wood, 1973).

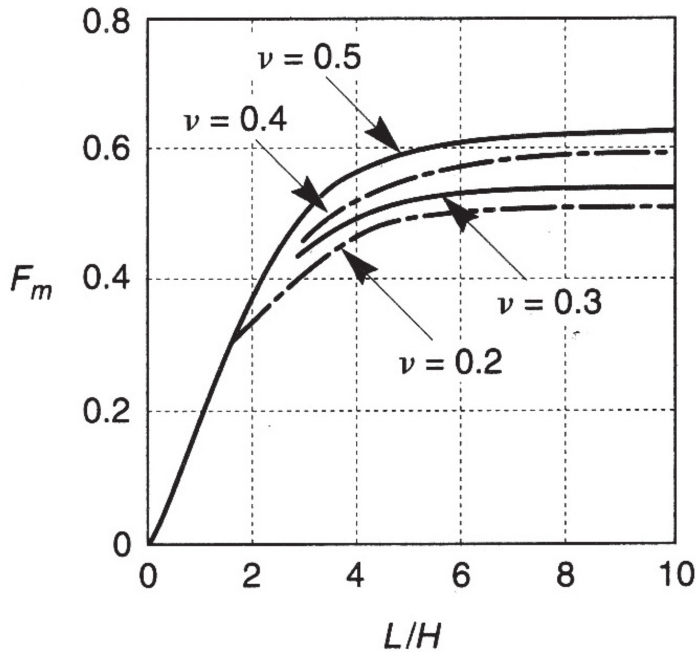


Figure I-5 Dimensionless moment factor for various geometries and soil Poisson's ratio values, *ν* (Wood, 1973).

I.3.3 Displacement-Based Analysis Methods

Analyses of retaining wall displacements can be performed using a procedure that accounts for the time-varying nature of the seismic excitation of the mass. Newmark (1965) developed such a procedure by recognizing that displacements accrue in a slope as a result of increments of time during which the seismic excitation causes the factor of safety to drop below one. Such procedures are discussed in Appendix H.

Richards and Elms (1979) proposed a method to determine the seismic displacement of gravity retaining walls. The procedure utilizes earth pressures calculated from the Mononobe-Okabe method but neglects factors such as the dynamic response of the backfill, kinematic factors, tilting mechanisms that cause the wall to rotate, and vertical accelerations. Displacements are based on Newmark analysis described in Franklin and Chang (1977) where an upper-bound, straight-line approximation is utilized to estimate displacements. In this methodology, failure is assumed to occur via sliding along the base of the wall. Elms and Richards (1990) adapted Whitman and Liao's (1984, 1985) statistical model for mean permanent movement of a sliding block. The statistical model shows that permanent displacements are lognormally distributed with a mean value of:

$$\bar{d}_{perm} = 37 \frac{v_{\max}^2}{PHA} \exp\left(\frac{-9.4 a_y}{PHA}\right) \quad \text{(I-8)}$$

where *PHA* is the peak horizontal ground acceleration (as used above), $v_{\max}$ is the peak ground velocity, and a_y is the yield acceleration that causes initiation of sliding along the base of the wall. The yield acceleration is derived from horizontal and vertical force equilibrium for the retaining wall and is expressed as:

$$a_y = \left[\tan \varphi_{backfill} - \frac{P_{AE} \cos(\delta + \theta) - P_{AE} \sin(\delta + \theta)}{W} \right] g \qquad \text{(I-9)}$$

where W is the weight of the wall.

As with the Newmark analysis, the issues discussed in Appendix H are applicable to interpreting the results of this analysis.

MSE slopes and walls are generally treated as rigid structures for seismic design purposes (Nova-Roessig and Sitar, 1996). Pseudo-static analysis is carried out assuming rigid behavior of the reinforced soil. However, field and model studies (Collin et al., 1992; Reinforced Earth Co., 1994; Richardson and Lee, 1974; Nagel, 1985; Fairless, 1989; Sakaguchi et al., 1992, Sakaguchi, 1996; Sugimoto et al., 1994; Nova-Roessig and Sitar, 1998) have shown that these structures are flexible and do not respond rigidly under seismic loading. The behavior lies somewhere between that of traditional retaining walls and unreinforced slopes. Displacement-based analysis specific to MSE walls and slopes are not routinely used. Nova-Roessig (1999) has proposed a deformation-based analysis for MSE walls and slopes similar to the simplified procedure for estimating earthquake-induced deformations in dams and embankments proposed by Makdisi and Seed (1978), but the procedure is limited to slopes having clean, cohesionless backfill material and lightweight facing panels.

Seismic compression has been observed both within the reinforced and unreinforced backfill (Murata et al., 1994; Nova-Roessig and Sitar, 1998). Methods described in Appendix G can be used to estimate the amount of seismic compression since no methods are currently available specific to MSE walls.

Typical static design of retaining walls assumes an active or passive condition within the soil mass (typically within the backfill) and mobilization of shear strength along a slip surface. The amount of wall displacement is typically small when this condition is reached. Therefore, for a static FS > 1, small displacements of retaining walls are anticipated and in most circumstances, particularly in residential construction, are acceptable. In a Mononobe-Okabe pseudo-static stability analysis, retaining walls with $FS > 1$ are unlikely to have significantly displaced during earthquake shaking and walls with $FS < 1$ may have had some displacement; however, the methodology does not quantify displacements. In contrast, the calculated displacement described in this section should be recognized as an estimate of the retaining wall performance but does not necessarily correspond to the actual displacement.

The calculated retaining wall displacements described in this section provide a tool for evaluation of whether retaining wall movements likely occurred at the site. If calculated displacements are zero with reasonable assumptions of ground motion and soil strength parameters, then earthquake-induced retaining wall displacements were unlikely to have occurred. Nonzero calculated displacements suggest wall movements were possible, especially if corroborated by field observations of distress to the retaining wall. Nonetheless, it should always be remembered that the field condition of the wall, backfill (e.g., presence of ground cracking), and improvements, if any, in front of the wall are the best indicators of movement of the wall. Such observations, if available, are more reliable than the analyses described above to establish whether wall movements may have occurred during an earthquake.

APPENDIX J

Investigation Checklists

This appendix compiles the investigation checklists from Chapter 3 into one compact set, in order to facilitate their reproduction for use in the field. However, as noted in Chapter 3, each checklist records only the most basic information: which element types are present at the house, which categories of damage were observed, and which areas were not investigated. A more complete field tool could be based on these checklists, but it might also provide space for recording notes, sketches, and quantities to support a repair scope, as well as separate rows for different subsets of similar elements, such as the perimeter walls on each side of the house.

Investigation Checklist 3-1 Permanent Ground Deformation

		Earthquake Damage				
Check if Present	*Element Type*	*Yes*	*Suspected*	*No*	*Non-EQ Damage*	*Not Investigated*
☐	Ground supporting the house	☐	☐	☐	☐	☐
☐	Ground outside the house footprint	☐	☐	☐	☐	☐

Investigation Checklist 3-2 Foundations, Slabs-on-Grade, and Basement Walls

		Earthquake Damage				
Check if Present	*Element Type*	*Yes*	*Suspected*	*No*	*Non-EQ Damage*	*Not Investigated*
☐	Slabs-on-grade *Repair threshold reminders:* • *Crack 1/8 inch, 1/2 inch* • *Crack offset 1/16 inch* • *Drop or slope 1/2 inch in 8 feet, 2 inches in 8 feet* • *Loss of bearing*	☐	☐	☐	☐	☐
☐	Footings or stemwalls, concrete *Repair threshold reminders:* • *Crack 1/8 inch, 1/2 inch* • *Crack offset 1/16 inch, 1 inch* • *Drop or slope 1/2 inch in 8 feet, 2 inches in 8 feet* • *Tilt or lean 1/4 inch per foot* • *Loss of bearing*	☐	☐	☐	☐	☐

Investigation Checklist 3-2 Foundations, Slabs-on-Grade, and Basement Walls (continued)

Check if Present	Element Type	Earthquake Damage: Yes	Earthquake Damage: Suspected	Earthquake Damage: No	Non-EQ Damage	Not Investigated
☐	Footings or stemwalls, masonry *Repair threshold reminders:* • *Crack in brick or masonry unit 1/8 inch, 1/2 inch* • *Crack in mortar joint 1/8 inch, 1/2 inch* • *Crack offset 1/16 inch* • *Drop or slope 1/2 inch in 8 feet, 2 inches in 8 feet* • *Tilt or lean 1/4 inch per foot* • *Loss of bearing*	☐	☐	☐	☐	☐
☐	Posts and piers *Repair threshold reminders:* • *Post lean 1/2 inch total over post height* • *Post lean 1/4 inch per foot over height* • *Floor drop 1/16 inch per foot and 1/4 inch total in 8 feet* • *Floor drop 1/4 inch per foot and 2 inches total in 8 feet* • *Deep pier shift 2 inches*	☐	☐	☐	☐	☐
☐	Basement walls, concrete *Repair threshold reminders:* • *Crack 1/8 inch, 1/2 inch* • *Crack offset 1/16 inch* • *Drop or slope 1/2 inch in 8 feet, 2 inches in 8 feet* • *Tilt or lean 1/4 inch per foot of height* • *Loss of bearing*	☐	☐	☐	☐	☐
☐	Basement walls, masonry *Repair threshold reminders:* • *Crack 1/8 inch, 1/2 inch* • *Crack offset 1/16 inch* • *Drop or slope 1/2 inch in 8 feet, 2 inches in 8 feet* • *Tilt or lean 1/4 inch per foot of height* • *Loss of bearing*	☐	☐	☐	☐	☐

Investigation Checklist 3-3 Sill Plates and Anchorage

Check if Present	Element Type	Earthquake Damage: Yes	Earthquake Damage: Suspected	Earthquake Damage: No	Non-EQ Damage	Not Investigated
☐	Wood sill plates *Repair threshold reminders:* • *Slide 1/4 inch* • *Fracture at multiple adjacent bolts* • *Splitting away from bolts* • *Splitting of retrofit block* • *Crushing 1/8 inch*	☐	☐	☐	☐	☐
☐	Steel cast-in-place anchor bolts *Repair threshold reminders:* • *15-degree angle of bolt from vertical* • *Gap 1/4 inch between bolt shank and sill plate* • *Spall 1-inch deep at anchor bolt*	☐	☐	☐	☐	☐
☐	Steel post-installed anchor bolts *Repair threshold reminders:* • *15-degree angle of bolt from vertical* • *Gap 1/4 inch between bolt shank and sill plate* • *Spall 1/2-inch deep at anchor bolt*	☐	☐	☐	☐	☐
☐	Steel post-installed plate connectors *Repair threshold reminders:* • *15-degree angle of bolt from vertical* • *Gap 1/4 inch between bolt shank and sill plate* • *Splitting of sill plate with 20% capacity loss in fasteners* • *Spall 1/2-inch deep at anchor bolt*	☐	☐	☐	☐	☐
☐	Hillside houses: framing-to-foundation connection *Repair threshold reminders:* • *At uphill connection, shift 1/2 inch of superstructure relative to foundation*	☐	☐	☐	☐	☐

Investigation Checklist 3-4 Wood-Frame Walls

Check if Present	*Element Type*	*Earthquake Damage*			*Non-EQ Damage*	*Not Investigated*
		Yes	*Suspected*	*No*		
☐	Horizontal wood siding *Repair threshold reminders:* • *Gap between siding and framing 1/4 inch* • *Split stud, loose nails, or torn building paper*	☐	☐	☐	☐	☐
☐	Plywood panel siding *Repair threshold reminders:* • *Gap between siding and framing* • *Panel rotation 1/4 inch from vertical* • *Nail tear through siding edge* • *Split stud or torn building paper*	☐	☐	☐	☐	☐
☐	Stucco *Repair threshold reminders:* • *Crack 1/64 inch, 1/8 inch* • *Crack offset 1/16 inch* • *Extensive crack pattern* • *Stucco spall* • *Wire fracture* • *Separation from sheathing or framing 1/8 inch*	☐	☐	☐	☐	☐
☐	Plaster on wood lath *Repair threshold reminders:* • *Crack 1/32 inch, 1/8 inch* • *Extensive crack pattern*	☐	☐	☐	☐	☐
☐	Plaster on gypsum lath *Repair threshold reminders:* • *Crack 1/64 inch, 1/8 inch* • *Shifting and detachment of lath*	☐	☐	☐	☐	☐
☐	Gypsum wallboard *Repair threshold reminders:* • *Crack 1/64 inch, 1/8 inch* • *Nail pop* • *Fastener tear-out* • *Slotting of back face at fasteners 1/2 inch* • *Panel visibly rotated in-plane* • *Detachment from framing*	☐	☐	☐	☐	☐
☐	Wood structural panel [(1)], particleboard, or fiberboard sheathing ☐ Assumed ☐ Confirmed *Repair threshold reminders:* • *Nail tear through panel edge* • *Gap between panel and framing* • *Split framing*	☐	☐	☐	☐	☐

Investigation Checklist 3-4 Wood-Frame Walls (continued)

Check if Present	*Element type*	*Earthquake Damage* *Yes*	*Suspected*	*No*	*Non-EQ Damage*	*Not Investigated*
☐	Diagonal lumber sheathing ☐ Assumed ☐ Confirmed *Repair threshold reminders:* • *Split boards at nails, limited v. extensive* *Fractured framing at nails*	☐	☐	☐	☐	☐
☐	Horizontal lumber sheathing ☐ Assumed ☐ Confirmed *Repair threshold reminders:* • *Split boards at nails, local v. extensive* • *Fractured boards, extensive* • *Gap 1/4 inch between board and framing* • *Split stud, loose nails*	☐	☐	☐	☐	☐
☐	Framing (2) *Repair threshold reminders:* • *Racking or leaning 1/2 inch, 2 inch in story* • *Slip or gap between elements (e.g., studs, plates, floor framing) 1/4 inch* • *Splitting, local v. extensive* • *Crushing 1/8 inch*	☐	☐	☐	☐	☐
☐	Tie-downs (3) ☐ Assumed ☐ Confirmed *Repair threshold reminders:* • *Splitting of post or stud, local v. extensive* • *Damage to fasteners* • *Spalling of concrete at tie-down*	☐	☐	☐	☐	☐
☐	Other load path elements (4) ☐ Assumed ☐ Confirmed *Repair threshold reminders:* • *Nail withdrawal 1/8 inch, 1/4 inch* • *Chord or collector splitting, local v. extensive*	☐	☐	☐	☐	☐
☐	Gravity-load-carrying elements *Repair threshold reminders:* • *Post lean 1/4 inch per foot of height* • *Relative shift 1/4 inch between post and supported member*	☐	☐	☐	☐	☐

(1) Wood structural panel includes plywood and oriented strand board.
(2) Includes wood studs, headers, blocking, and other wall framing members.
(3) Includes tie-down anchors, rods, and fasteners.
(4) Includes clips, straps, and other metal hardware.

Investigation Checklist 3-5 Other Seismic-Force-Resisting Elements

Check if Present	Element Type	Earthquake Damage: Yes	Earthquake Damage: Suspected	Earthquake Damage: No	Non-EQ Damage	Not Investigated
☐	Portal frames ☐ Assumed ☐ Confirmed *Repair threshold reminders:* • *Splitting of beam at beam-pier connection, limited v. extensive* • *Racking or leaning 1/2 inch in story, 2 inches in story*	☐	☐	☐	☐	☐
☐	Proprietary shear walls ☐ Assumed ☐ Confirmed *Repair threshold reminders:* • *Racking or leaning 1/2 inch in story, 2 inches in story*	☐	☐	☐	☐	☐
☐	Hillside houses: diagonal braces *Repair threshold reminders:* • *Fracture of wood members* *Fracture of rod braces or turnbuckles*	☐	☐	☐	☐	☐
☐	Steel moment frames or cantilevered columns ☐ Assumed ☐ Confirmed *Repair threshold reminders:* • *Racking or leaning 1/2 inch in 8-feet height, 2 inches in 8-feet height* • *Fracture of anchor bolts* • *Fracture at moment connection* • *Fracture at cantilevered column base*	☐	☐	☐	☐	☐

Investigation Checklist 3-6 Floors, Ceilings, and Roofs

Check if Present	Element Type	Earthquake Damage: Yes	Earthquake Damage: Suspected	Earthquake Damage: No	Non-EQ Damage	Not Investigated
☐	Floor, ceiling, or roof framing *Repair threshold reminders:* • *Loss of bearing 1/4 inch, 1 1/2 inches* • *Out-of-plane distortion 1/2 inch in 8 feet, 1 1/2 inches in 8 feet*	☐	☐	☐	☐	☐
☐	Floor diaphragms *Repair threshold reminders:* • *In-plane racking 1/2 inch in 8 feet, 2 inches in 8 feet*	☐	☐	☐	☐	☐

Investigation Checklist 3-6 Floors, Ceilings, and Roofs (continued)

Check if Present	*Element Type*	*Earthquake Damage*			*Non-EQ Damage*	*Not Investigated*
		Yes	*Suspected*	*No*		
☐	Ceiling diaphragms *Repair threshold reminders:* • *Gypsum board crack 1/64 inch, 1/8 inch* • *Plaster crack 1/64 inch, 1/8 inch* • *Plaster cracking or spalling, local v. extensive* • *In-plane racking 1/2 inch in 8 feet, 2 inches in 8 feet* *Slotting of back face at fasteners 1/2 inch*	☐	☐	☐	☐	☐
☐	Roof diaphragms *Repair threshold reminders:* • *In-plane racking 1/2 inch in 8 feet, 2 inches in 8 feet* • *Fracture of lumber sheathing*	☐	☐	☐	☐	☐

Investigation Checklist 3-7 Fireplaces and Chimneys

Check if Present	*Element Type*	*Earthquake Damage*			*Non-EQ Damage*	*Not Investigated*
		Yes	*Suspected*	*No*		
☐	Chimneys or fireboxes *Repair threshold reminders:* • *Lean 1/2 inch over 8-feet height, 2 inches over 8-feet height* • *Crack in brick 1/16 inch, 1/8 inch* • *Crack in mortar 1/8 inch*	☐	☐	☐	☐	☐
☐	Anchorage to house framing	☐	☐	☐	☐	☐
☐	Bracing to roof framing	☐	☐	☐	☐	☐
☐	Interior fireplace surrounds *Repair threshold reminders:* • *Veneer detached 1/2 inch*	☐	☐	☐	☐	☐

Investigation Checklist 3-8 Nonstructural Components

Check if Present	Component Type	Earthquake Damage			Non-EQ Damage	Not Investigated
		Yes	Suspected	No		
☐	Mechanical, electrical, or plumbing component bracing and anchorage	☐	☐	☐	☐	☐
☐	Architectural component bracing and anchorage	☐	☐	☐	☐	☐

Investigation Checklist 3-9 Appurtenances

Check if Present	Element Type	Earthquake Damage			Non-EQ Damage	Not Investigated
		Yes	Suspected	No		
☐	Appurtenances	☐	☐	☐	☐	☐
☐	Connections of appurtenances to house	☐	☐	☐	☐	☐

Glossary and Acronyms

K.1 Glossary

The definitions and descriptions provided here are intended for use with these *Engineering Guidelines* only. They may differ from definitions of similar or identical terms used in the *General Guidelines*, in building codes, or in other technical standards or references. Terms used in the *Engineering Guidelines* that are not included here should be understood to have the meanings ascribed to them by applicable building codes or regulations.

basement wall. A wall occurring in a basement, usually of concrete or masonry construction. See also stemwall, retaining wall, and foundation damage patterns.

crawlspace. An unfinished underfloor space that is not a basement. For a hillside house, the unfinished area under the first floor is often a full story or more in height at the downhill end.

cripple wall. A framed wall extending from the top of the foundation to the underside of the floor framing of the story above. Cripple walls commonly occur at the perimeter of a crawlspace.

consultant, geotechnical. A technical consultant specializing in earthquake-induced ground deformations. See Section 1.1.

consultant, structural. A technical consultant specializing in earthquake damage to structures. See Section 1.1.

consultant, technical. A consultant with specialized expertise who has been engaged by the user of the *General Guidelines*, typically to assess damage extending beyond the scope of that document.

damage, earthquake. An adverse, non-trivial, physical change in the safety, serviceability, appearance, or repairability of a component or portion of a building caused by earthquake ground shaking or earthquake-induced permanent ground deformation. See Section 1.2.

damage, worsened. Damage of increased severity or extent, such that the repair method identified for the damage pattern is different from that for the pre-earthquake damage pattern. An example is a crack in stucco that both widens and lengthens such that repair is required where it would not have been for the pre-earthquake crack.

damage, structurally significant. Earthquake damage resulting in a non-trivial, adverse change in the ability of the building or any of its elements to sustain load or resist future earthquake shaking, whether caused by structural or geotechnical damage mechanisms. See Section 1.2.

diagonal lumber sheathing. 1× nominal sawn lumber boards applied diagonally (at approximately 45 degrees) and nailed to supporting wall, roof, or floor framing members, with nominal gaps (commonly 1/4 inch to 1/2 inch) between boards.

drilled pier (caisson). A cast-in-place deep foundation element constructed by drilling a hole (with or without casing) into soil or rock and filling it with fluid concrete.

exterior insulation and finish system (EIFS). A nonstructural, non-load-bearing exterior wall cladding system that consists of an insulation board attached either adhesively or mechanically, or both, to the substrate; an integrally reinforced base coat; and a textured protective finish coat. The appearance of EIFS can be similar to stucco.

foundation damage patterns. The following are terms used in this document to describe foundation, slab-on-grade, and basement wall damage patterns.

- **loss of bearing.** For foundations and slabs-on-grade, a condition in which supporting soil under a portion of a slab or footing has experienced permanent ground deformation from any cause, leaving that portion of the slab or footing unsupported with respect to vertical loads. Also described as "undermining."
- **offset.** With respect to any flat surface, an out-of-plane difference in the surface location across a crack or joint, measured normal to the plane of the surface. Because offset refers to an out-of-plane dislocation, it is different from a crack, sliding, shearing, or racking in-plane; continuous deformation or bending; or uniform slope or settlement. See Figure K-1.
- **settlement.** For foundations and slabs-on-grade, downward movement of one portion of a slab or foundation relative to the adjacent portions. Such settlement can result in a vertical offset or a slope, or both. Uniform settlement of houses can often occur without damage; damage is generally caused by differential settlement. See Figure K-2.
- **slope.** For foundations and slabs-on-grade, a portion of a slab or foundation that is not level on one or both sides of a crack. See Figure K-3.
- **spreading.** For foundations and slabs-on-grade, movement of one portion of a slab or foundation away from the adjacent portion, either at locations of cracking or construction joints.

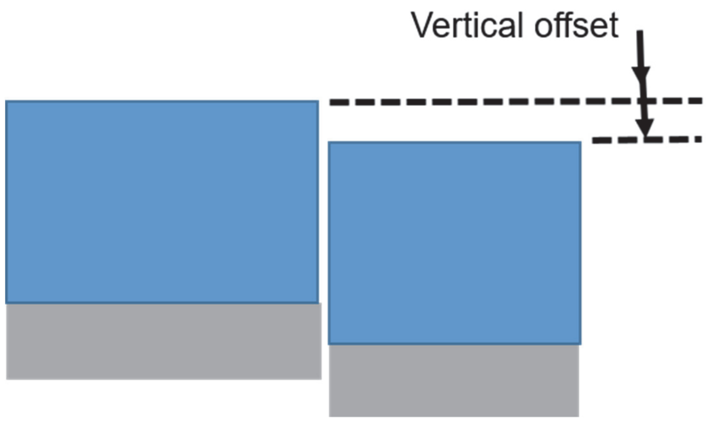


Figure K-1 Foundation or slab offset. Vertical offset shown, horizontal offset similar.

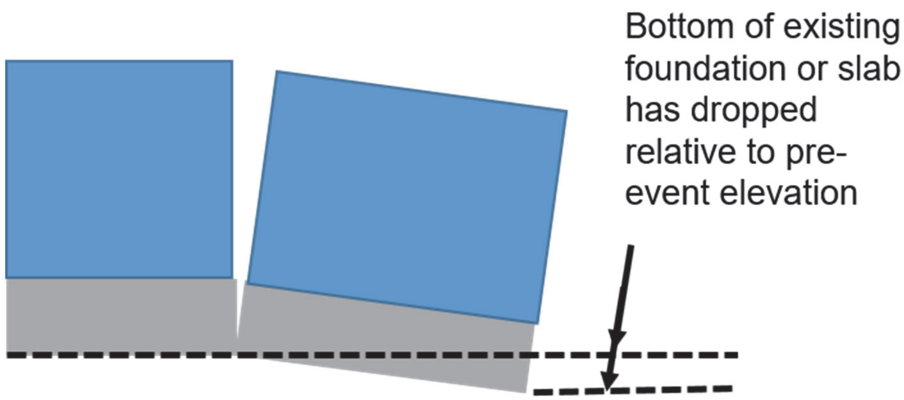


Figure K-2 Foundation or slab settlement.

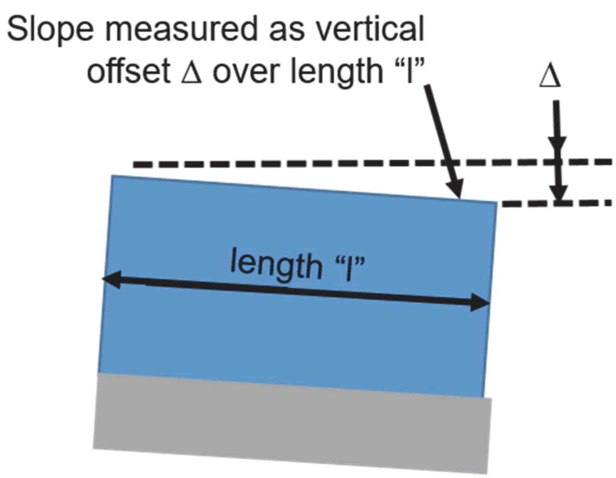


Figure K-3 Foundation or slab slope.

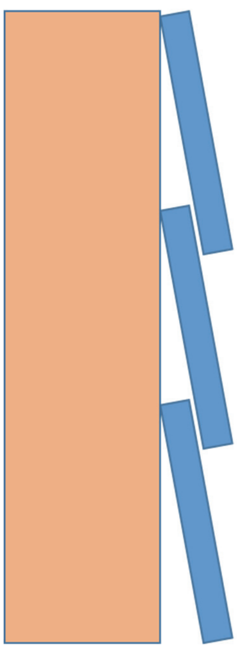
hillside house. A house constructed on a site with a slope of more than one vertical to five horizontal. See Section 2.2.3.

house. A detached dwelling of light wood-frame construction, typically with no more than two dwelling units.

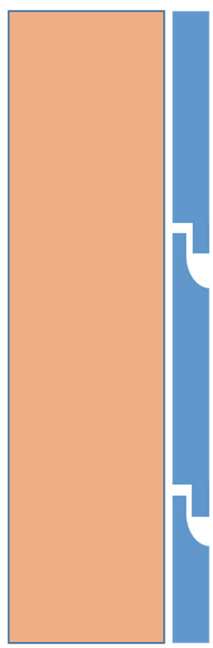
horizontal lumber sheathing. 1× nominal sawn lumber boards applied perpendicular and nailed to supporting wall, roof, or floor framing members, with nominal gaps (commonly 1/4 inch to 1/2 inch) between boards. Lumber sheathing with larger gaps (typically called spaced or skip sheathing) is not considered to be horizontal lumber sheathing by these *Engineering Guidelines*.

horizontal wood siding, lap siding. Horizontal wood lumber board siding that is installed at an angle to the wall framing, primarily serving as an exterior finish. This siding does not provide appreciable strength and stiffness to the seismic-force-resisting system. See Figure K-4a.

horizontal wood siding, shiplap siding. Horizontal wood lumber board siding in which the siding has rabbeted edges top and bottom, allowing it to sit flush against and be nailed to the supporting wall framing. This siding provides strength and stiffness to the seismic-force-resisting system. See Figure K-4b.



(a)



(b)

Figure K-4 Features of different types of horizontal wood siding: (a) lap siding is installed at an angle to the face of studs; and (b) shiplap siding is rabbeted so that it can sit flush against the face of studs.

heating, ventilation and air conditioning (HVAC) systems. Systems and equipment related to house heating, ventilation, and air conditioning.

leaning. Out-of-plane distortion, typically of a wall element. Leaning is generally caused by out-of-plane displacement of the top of the wall relative to the bottom of the wall.

living space over garage house. A house in which a primary occupied space occurs in an upper story that extends substantially or completely over a ground story constructed primarily as a garage, including utility and storage areas. See Section 2.2.5.

mechanical, electrical, and plumbing (MEP) systems. Systems and equipment related to house mechanical, electrical, and plumbing systems. Mechanical systems commonly include HVAC systems.

masonry veneer, adhered. Masonry veneer secured and supported through the adhesion of a bonding material to a backing.

masonry veneer, anchored. Masonry veneer secured with mechanical fasteners to a backing.

nail pop. A circular crack or slight bump, or both, over the head of a fastener in wall, ceiling, floor, or roof finishes or sheathing. Nail pops can be caused by the initial backing out of a fastener due to earthquake loading. Nail pops can also develop due to improper installation, framing shrinkage, and thermal and moisture cycling.

permanent ground deformation, earthquake-induced. Any earthquake-generated process that leads to deformations within a soil medium, which in turn results in permanent horizontal or vertical displacement of the ground surface.

plate connectors. Connectors between foundation sill plates and concrete or masonry foundations, also known as foundation anchors. These connectors are primarily used in repair and retrofit where there isn't enough overhead height to drill holes for installation of anchor bolts. Plate connectors can be of prescriptive design, as found in IEBC Appendix Chapter A3, or manufactured proprietary devices.

plywood panel siding. A wood structural panel product manufactured for use as an exterior finish material. Exterior appearance can be rough-sawn texture or with vertically oriented grooves.

portal frame. Engineered or prescriptive seismic-force-resisting elements consisting of narrow wood structural panel shear walls connected to a header or other continuous beam at the top of the wall piers. Portal frames are typically used around large wall openings.

retaining wall. A structure that resists the lateral pressure of soil on one side, typically used to establish a defined change of grade. In these *Engineering Guidelines*, retaining walls are separate from the house structure and are located outside the house footprint. A wall that serves the same purpose as a retaining wall but is part of the house structure should be considered a stemwall or a basement wall, depending on its height.

sill plate (foundation sill plate). A framing member bearing on the top of a foundation and providing support to the structure above.

slab-on-grade. A concrete slab, typically used as a floor or as pavement, that is continuously supported by the underlying soil. A slab-on-grade is distinguished from a slab that spans between supporting elements. A slab-on-grade can have a thickened edge cast monolithically with the slab or can be cast over an edge footing (typically at its perimeter) as "two-pour construction." See also foundation damage patterns.

stemwall. The vertical extension of a footing, forming the walls of a crawlspace. Stemwalls typically extend to the underside of the first-floor framing. By contrast, a *framed* wall extending from the top of the foundation to the underside of the floor framing of the story above is a cripple wall. A basement wall is similar to a stemwall, but for a full basement story.

racking. in-plane distortion, typically of a wall element. Racking is generally caused by in-plane displacement of the top of the wall relative to the bottom of the wall, but can also be caused by in-plane vertical displacement of one end of a wall relative to the other end.

upgrade, code-triggered. A modification to the building or any of its elements with respect to the pre-earthquake condition that is required by code provisions or other regulations. See Appendix A.

upgrade, voluntary. A modification to the building or any of its elements with respect to the pre-earthquake condition that is not required by code provisions or other regulations. See Appendix B.

weather resistive barrier (WRB). A moisture barrier commonly provided under exterior finish materials, such as roofing and siding. WRB includes building paper and a range of other product types.

wood structural panel (WSP) sheathing. Plywood or oriented strand board (OSB) panels used as sheathing for shear walls or diaphragms.

K.2 Acronyms

ATC	Applied Technology Council
CGS	California Geological Survey
CMU	concrete masonry unit

CUREE	Consortium of Universities for Research in Earthquake Engineering
EERI	Earthquake Engineering Research Institute
EIFS	exterior insulation and finish system
HVAC	heating, ventilation, and air conditioning
IBC	International Building Code
IEBC	International Existing Building Code
IRC	International Residential Code
MEP	mechanical, electrical, and plumbing
MMI	Modified Mercalli Intensity
NISEE	National Information Service for Earthquake Engineering, University of California at Berkeley
OSB	oriented strand board
PEER	Pacific Earthquake Engineering Research Center
USGS	U.S. Geological Survey
WSP	Wood structural panel

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