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CHAPTER 4

GROUND FAILURE DURING EARTHQUAKES

By

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4.1 Introduction

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

- **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 5 to 20 km. 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 10 m. When fault slip extends to the ground surface, the resulting ground displacements are termed “surface fault rupture.” Examples of California earthquakes with surface fault rupture include 1906 San Francisco, 1971 San Fernando, 1992 Landers, and 1999 Hector Mine.

- **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). Herein, the initiating disturbance is assumed to be cyclic shear deformations resulting from an earthquake. The loss of shear strength associated with liquefaction can create ground deformations and/or instability. 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. 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. If the post-liquefaction strength exceeds the static shear stress, the ground may “lurch” during strong pulses of motion when the shear strength is temporarily exceeded, a condition termed *cyclic mobility*. 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.
• **Seismically-induced landsliding** Inertial forces generated by strong shaking of earth slopes can cause transient shear stresses\(^1\) 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\(^2\) within the slope materials and is referred to as seismically-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. Note that liquefaction-induced lateral spreading is one example of landsliding, although the analysis of lateral spread displacements are generally evaluated through a different set of procedures than are used for conventional landsliding.

• **Seismic compression** Unsaturated soil subject to large transient shear stresses can experience volumetric strains, which results in ground surface settlements and potential lateral movements (near slopes). This process is termed seismic compression and has been observed to be especially prevalent in artificial fill soils.

When considering whether the above modes of ground failure may have affected a site, it is important to recognize the requisite conditions for their occurrence. With surface fault rupture, the requisite condition 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 break. Soil liquefaction requires the presence of ground water, soil materials considered susceptible to liquefaction (generally sands, gravels, and low plasticity silts at low densities), and dynamic loading of sufficient amplitude and duration to trigger liquefaction in those materials. 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. Seismic compression requires relatively strong shaking and unsaturated soil.

If all sites were pristine and stable in the absence of earthquakes, identification of seismically-induced ground failure during a post-earthquake site reconnaissance 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 4.8 and include:

• **Consolidation settlement:** Volume change due to dissipation of excess pore pressure\(^3\) resulting in expulsion of water from the soil matrix and increased effective stress. The rate of settlement is dependent upon soil properties and the length of the drainage path. The excess pore pressures responsible for consolidation may result from changes in overburden pressure (i.e., fill placement, addition of structural loads) or changes in ground water levels.

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\(^1\) Transient shear stresses are time varying stresses that are present during shaking, but are not present upon the conclusion of shaking.

\(^2\) Permanent shear deformations are ground deformations that remain upon the conclusion of earthquake shaking.

\(^3\) Excess pore pressure is defined as pore pressures beyond the hydrostatic pore pressure.
• **Immediate settlement:** Settlement caused by small-strain shear and/or volumetric deformations in soil that are not associated with consolidation or hydrocompression. 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, but rather, depends principally on the bulk and shear modulus of the soil.

• **Hydro-compression settlement:** Volume reduction of unsaturated soils upon wetting, which is associated with collapse of the soil fabric. Soils subject to collapse can include wind-deposited sands and silts, alluvial fan and mudflow sediments, and some man-made fills. 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.

• **Expansive soil movement:** Shrink/swell of plastic clays when the water content is reduced (drying) or increased (wetting). 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. A good indication that expansive soils are present at a site is desiccation cracks in the soil surface.

• **Landsliding:** The movement of a mass of rock, debris, or earth down a slope. 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 changes in slope geometry (i.e., excavation near slope toe), loading of the top of slope, and increased water pressure within the slope.

• **Slope creep:** Slow downslope movement of plastic rock and soil materials. The rate of creep is dependent on factors such as material type, slope inclination, and water content fluctuations within the slope. Slope creep occurs within shallow soil/rock materials, and hence damage from slope creep is generally confined to areas along a slope face or near the top of slope.

• **Retaining walls failure:** Tilt, deterioration, and failure of retaining walls. 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).

During post-earthquake reconnaissance of a flat site, care should be exercised to distinguish ground settlements and/or heave that are characteristic of static volume change phenomena from ground settlements associated with liquefaction or seismic compression. Likewise, post-earthquake
reconnaissance of sloping sites must distinguish long-term slope instability (landslides), creep, or retaining wall movements from ground deformations associated with seismically-induced landsliding.

It is important to recognize a hybrid failure mode that is combination of non-seismic soil movement and seismic shaking of improvements that is often misidentified as earthquake-induced soil movement. This mode of damage occurs when improvements on the surface (typically non-structural 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 may 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. A classic example of this mode of failure was the floor slab of Frank Lloyd Wright’s Hanna House on the Stanford campus during the Loma Prieta Earthquake as observed by Burton (2004, personal communication). 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 non-structural 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.

The following section provides general guidelines for post-earthquake reconnaissance to evaluate the potential occurrence of seismically-induced ground failure phenomena. The emphasis is on the issues raised above – distinguishing seismic ground failure phenomena from pre-earthquake ground deformations caused by long-term (non-seismic) geotechnical processes. Subsequent sections are focused on specific mechanisms of seismic and non-seismic ground failure. Presented in those sections are methods that can be used to assess whether a particular type of ground failure may have occurred and to estimate the resulting ground displacements. Repair and mitigation strategies are also discussed.
4.2 Reconnaissance of Ground Failure

An investigation of whether seismically-induced ground failure occurred at a site begins with a thorough site reconnaissance. An essential part of a reconnaissance is examination of the general vicinity of the site, the site itself, and improvements on the site for indications of possible seismically-induced ground failure. 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. The level of effort during a reconnaissance will vary depending on how readily observable and unambiguous the indicators of seismically-induced ground failure are at a site.

Geotechnical investigations involving subsurface exploration, laboratory testing, and engineering analyses are only needed under the following circumstances:

1. Clear indicators of seismically-induced ground failure are present at the site. In this case, geotechnical investigations and analyses may be needed to determine the nature and extent of the seismically-induced ground failure and design appropriate repairs.

2. Indicators of seismically-induced ground failure are present in the immediate vicinity of the site. In this case, the investigations and analyses are needed to determine if ground failure occurred at the site.

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

The central elements of a site reconnaissance are discussed in the remainder of this section. Examples of seismically-induced ground failure (Table 4.1 on page 12) and indicators of those ground failure modes (Table 4.2 on page 14) are also discussed.

4.2.1 Examples of Seismically-Induced Ground Failure

Investigators should be familiar with and be able to recognize the modes of seismically-induced ground failure. In order to facilitate recognition of seismically-induced ground failure, examples of the modes of failure are presented Table 4.1 on page 12. The table contains a brief description of each mode (e.g., surface fault rupture, liquefaction, landsliding, seismic compression, retaining wall failure).

4.2.2 Site Investigation

A site-specific investigation of possible earthquake-induced ground failure 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 “regional” reconnaissance of the area surrounding the subject site and “site specific” reconnaissance. Each component of the site investigation is discussed below.
1. **Literature Review**  For post-earthquake investigation of possible earthquake-induced ground failure, the scope of the office study 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 (i.e., tagging and first hand intensity reports). Sources of this information may include web-based reports (e.g., USGS, SCEC, GEES, web pages of Civil Engineering departments at local universities) and post-earthquake reconnaissance reports by organizations such as the Earthquake Engineering Research Institute (EERI). Typically included within these publications are maps showing where ground failure was and was not observed. 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.

2. **Field Reconnaissance**  The principal objective of field reconnaissance is to identify features that may indicate seismically-induced ground failure or static ground deformation modes at a site of interest. Field observations for a site reconnaissance occur primarily on two scales: regional and site specific. In the following, the objectives of each are described along with the ground deformation features that investigators should look for.

4.2.2.1 **Regional Reconnaissance**

The principal purpose in performing a regional survey is to identify damage patterns and ground deformation features in the surrounding area that may provide insight into ground failure modes that may have affected the specific site under investigation. In particular, an investigator performing regional reconnaissance seeks to (1) identify patterns of damage and ground deformation that establish whether or not ground failure occurred in the area and (2) establish a baseline earthquake intensity (e.g. using MMI scale) for the area.

Regional reconnaissance is typically performed by driving and walking through the neighborhood of the property. During the reconnaissance, obvious indicators of ground failure should be noted, if present, within several blocks of the house. Such indicators might include:

- surface fault rupture (e.g., Figure 6 and Figure 7),
- large lateral or vertical ground displacements associated with liquefaction, such as flow failure (e.g., Figure 37) or lateral spreading (e.g., Figure 23),
- sand boils from liquefaction (e.g., Figure 26),
- landsliding, as evidenced by rockfalls (e.g., Figure 32 to Figure 35), linear fissures in ground parallel to the strike of a slope (not to be confused with fissuring caused by desiccation of expansive soil, e.g., Figure 36), or rising or bulging of the ground surface near the toe of slope.
If the above obvious indicators of ground failure are not present, then detailed regional reconnaissance should be undertaken to investigate the potential for more subtle ground failure involving relatively small ground displacements. During such reconnaissance, the condition of the following should be noted, as applicable:

1. **Streets, curbs, and sidewalks** Because they are relatively brittle and cover large areas, streets, curbs, and sidewalks are excellent indicators of ground failure. Unusual cracking of these improvements (i.e., cracking not associated with shrinkage stresses, thermal stresses, traffic patterns, effects of vegetation like large trees (Figure 55) are indications of ground failure. For example, evidence of conspicuous and unusual damage to the streets, driveways, sidewalks, curbs, or gutters near the residence may indicate that ground failure occurred during the earthquake (e.g., Figure 53, Figure 56 to Figure 70). An apparent random pattern of damage to these improvements, especially if the cracks are not fresh, may indicate that non-seismic deformation modes such as expansive soils or collapsible soils may be present and resulting in ground movement.

2. **Utility lines** Utility lines can be sensitive to ground deformations, and breaks in these lines are also a good indicator of ground failure. Examples of these utilities include water service pipes, water mains, sanitary sewer pipes, and gas pipes (e.g., Figure 7, Figure 22, and Figure 71). 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).

3. **Nearby residences** Damages to nearby properties are good indicators of the intensity of shaking experienced at the subject property. The condition of improvements with different vulnerabilities to earthquake damage should be noted. For example, damage to utility lines (e.g., Figure 7, Figure 22, and Figure 71) flatwork (e.g., Figure 56 to Figure 68), or foundations (e.g., Figure 18, Figure 19, Figure 42, Figure 90, and Figure 91) at nearby residences are indicators of ground failure that warrant consideration at the subject site. Block wall damage is common even at low intensity of earthquake shaking and, is not a good indicator of seismically-induced ground failure if the crack pattern in the wall is not consistent with a prevailing pattern of ground deformation (e.g., Figure 72 and Figure 73).

Observations from the regional survey should be documented with photographs, maps, and notes, and should accurately reflect the condition of the surrounding area. Therefore, indications of both earthquake-related ground failure and non-earthquake damage should be noted.

### 4.2.2.2 Site Specific Reconnaissance

The objectives of site specific reconnaissance are to (1) evaluate whether regional ground failure mechanisms observed in the surrounding area during regional reconnaissance are manifest at the property of interest, (2) evaluate whether localized ground failure with a unique character relative to the surrounding area may have occurred at the site, and (3) to identify additional investigation activities (i.e., geotechnical investigation) that may be needed to definitively identify the ground failure mechanisms, and if necessary, to facilitate the design of mitigation measures.
Investigators performing site-specific reconnaissance 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 (e.g., Figure 74). In particular, an investigator should be familiar with the common foundation types utilized for residential structures in California (e.g., Figure 75 to Figure 78 and discussed in Chapter 5). Figure 79 presents typical inspection points for a residential structure that assist in evaluating whether ground failure may have caused damage to the structure. In addition, minimum guidelines for inspection of geotechnical hazards may be found in ATC 20 (1994).

The tasks associated with a site-specific field reconnaissance should include the following:

1. Discuss the pre- and post-earthquake condition of the property with the owner/occupant and report, based on their recollection, those damage features that pre-existed the earthquake as well as 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 ground failure 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 ground cracks. 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 re-leveling compounds, etc. should be noted, if present.

While performing Tasks 2 and 3, the investigator should report any obvious evidence of ground failure such as manifestations of settlement (e.g., Figure 18, Figure 19, Figure 28, Figure 40, Figure 41, Figure 42, Figure 88, Figure 89, and Figure 91), liquefaction (e.g., Figure 26 and Figure 27), obvious evidence of slope movement such as linear fissures in ground parallel to a slope (again, not to be confused with fissuring caused by desiccation of expansive soil) (e.g., Figure 53, Figure 54, Figure 94, and Figure 95), rising or bulging of the ground surface, or collapse/significant rotation of retaining structures (e.g., Figure 46 to Figure 50).

Whether or not the above obvious indicators of ground failure are observed, the inspection report for the property should document the condition of the following components of the site:

1. **Concrete flatwork (driveway, patios, sidewalks)** Fresh cracks in flatwork or the opening of construction joints are good indicators of seismic ground failure, especially if the cracking pattern is consistent with a prevailing pattern of ground deformation (e.g., Figure 60 to Figure 64, Figure 66, Figure 68, Figure 69, and Figure 70). Also, fresh-appearing out-of-levelness in patios, decks, etc. not consistent with the normal construction practices (e.g., patios adjacent to structures are sloped away from the building for drainage) are indicators of possible seismically-induced ground failure. Fresh cracks that form a random pattern may represent pre-earthquake damage that was exacerbated by the earthquake strong shaking. Indicators such as paint or debris within cracks, previous patching of cracks, previous
releveling, etc. are good indicators of pre-earthquake damage possibly associated with static
ground deformation (e.g., Figure 80 to Figure 86).

2. **Utility lines** Damage to underground utility lines on the property such as water service lines,
sanitary sewer lines, gas lines are indicators of ground failure (e.g., Figure 7, Figure 22, and
Figure 71). The condition of these lines, if not obviously damaged, and serviceability of the
lines (i.e., recent clogging of sewer lines) should be queried during discussions with people
most knowledgeable about the post-earthquake condition of the property.

3. **Swimming pool** Swimming pools can be an effective means by which to assess the levelness
of at-grade improvements (e.g., Figure 74). Typically, swimming pool waterline tile are
installed reasonably level and significant deviations from level may be indications of earth
movement, seismically-induced or otherwise. Fresh-appearing out-of-levelness of the pool
coping not consistent with construction tolerances and fresh appearing cracking of the pool
shell are indicators of possible seismically-induced ground failure. 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 whether swimming
pool deformations occurred prior to or as a result of the earthquake.

4. **Foundations** Indicators of seismically-induced ground failure related to foundation
performance include the following: (1) fresh cracking in concrete foundation elements (e.g.,
Figure 12, Figure 90 to Figure 93), (2) soil deformations adjacent to foundations (e.g.,
bulging or tension cracks) (e.g., Figure 18, Figure 19, Figure 21, Figure 28, Figure 38 to
Figure 41, Figure 88, Figure 89, Figure 92, and Figure 93), and (3) out-of-levelness
consistent with conspicuous earthquake damage to adjacent house walls (e.g., Figure 29 to
Figure 30, and Figure 42).

Indicator 1 (cracking) requires direct inspection of foundation elements. This may be
possible 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, investigators should look for fresh cracks in tile applied on a slab-on-
grade surface or fresh cracks in wall finishes (stucco or drywall) above the foundation
elements. Fresh cracking in any of these elements indicates possible seismically-induced
ground failure. Conversely, patching of cracks, debris within cracks, and evidence of
previous releveling are indicators of long-term, re-occurring ground movement unrelated to
earthquake shaking (e.g., Figure 81, Figure 82, Figure 86, and Figure 87).

Indicator 2 (soil failure near foundation) involves ground failure that is localized around
foundations, indicating full or partial bearing failure or foundation sliding (e.g., Figure 12,
Figure 13, Figure 18, Figure 19, and Figure 20). This mode of ground failure usually results
from seismic pore-pressure induced strength loss in foundation soils (e.g., liquefaction), and
in past earthquakes has generally only been observed in structures with significant bearing
loads (typically $\geq 3$ stories in height).

Indicator 3 (out-of-level floors) is 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 ground failure 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 and expansive soil), indicators of which
may or may not have been exacerbated by the earthquake (e.g., slope creep and soil
expansion are conditions unaffected by earthquake ground shaking).

It should be noted that Indicators 1 and 3 (i.e., cracking and unlevelness of floors) are present
in virtually every foundation. Accordingly, care must be exercised to distinguish possible
seismic ground failure damage from pre-earthquake conditions. In some cases, the
distinction will not be clear, particularly if reconnaissance is performed many months or
years following the earthquake. In such cases, further analysis will be necessary to determine
the likely cause of the observed damages. Procedures for performing such analyses are
described in subsequent sections of this chapter.

5. \textit{Residence finish} Finish elements that should be inspected and documented include wall
finishes (stucco, drywall), masonry block walls, and chimneys. All of these elements tend to
be vulnerable to shaking-induced damage. Hence, the presence of damage to such elements
is not necessarily indicative of ground failure. To assess the possible contribution of ground
failure to damage in these elements, a pattern of ground deformations should be present
based on inspections of other components of the site 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 extremely unlikely that seismically-induced ground failure
occurred (e.g. Figure 72 and Figure 73).

Once damage to all of the above elements has been documented, the investigator should look for
patterns in the site performance (e.g., cracking, unlevelness) to evaluate whether the damage is
indicative of a large ground failure mechanism (i.e., slope deformation, differential settlement across
a fill pad) or is a localized phenomenon. Random cracking that does not form a pattern is likely not
associated with ground failure.

If clear evidence of seismic ground failure is found, the final objective of the reconnaissance is to
identify additional investigation activities (i.e., geotechnical investigation) that will be needed to
identify the specific mechanism of ground failure and to facilitate the design of mitigation measures.
Similarly, if evidence of ground failure is found, but it cannot be definitively attributed to either a
static or seismic damage mode, the investigator should identify additional investigation activities
that will be needed to identify the mode of damage.
Table 4.1 Summary of Seismically-induced Ground Failure Modes and Examples

<table>
<thead>
<tr>
<th>Seismically-induced Ground Failure Mode</th>
<th>Description</th>
<th>Examples</th>
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<tr>
<td>Fault Rupture (Section 4.3)</td>
<td>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 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. 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). 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. Distributed faulting is discontinuous in nature and occurs over a zone that can 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).</td>
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<td>Liquefaction (Section 4.4)</td>
<td>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). 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 at-grade improvements. Consequences of liquefaction can be grouped into the general categories of flow failure and cyclic mobility. 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. Resulting shear deformations are typically large (i.e., large translational or rotational failures) and often occur shortly after the conclusion of earthquake shaking. Cyclic mobility occurs when the post-liquefaction shear strength is greater than the static shear stress. Accordingly, 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.</td>
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<td></td>
<td></td>
<td>Figure 30</td>
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<tr>
<td></td>
<td></td>
<td>Figure 31</td>
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<td></td>
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<td>Figure 37</td>
</tr>
<tr>
<td>Seismically-induced Ground Failure Mode</td>
<td>Description</td>
<td>Examples</td>
</tr>
<tr>
<td>----------------------------------------</td>
<td>-----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------</td>
<td>---------------------------------</td>
</tr>
</tbody>
</table>
| Seismically-induced Landsliding (Section 4.5) | Seismically-induced landslides involve permanent shear deformations within geologic media. Landslides can be subdivided into several generalized categories:  
1. Masses of disrupted slide material, such as rock falls or avalanches.  
2. Relatively coherent slide masses whose displacement is accommodated along well-defined slip surfaces or across relatively broad, distributed shear zones.  
3. Lateral spreads and flows associated with soil strength loss due to pore pressure increase. | Figure 31, Figure 32, Figure 33, Figure 34, Figure 35, Figure 36, Figure 37 |
<p>| Seismic Compression (Section 4.6)       | 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/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. | Figure 38, Figure 39, Figure 40, Figure 41, Figure 42 |
| Retaining Wall Failure (Section 4.7)    | Retaining wall failure is defined as excessive permanent deformation of the retaining structure. The deformations may consist of sliding, rotation, bending, settlement, etc., 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 motions. | Figure 43, Figure 44, Figure 45, Figure 46, Figure 47, Figure 48, Figure 49, Figure 50 |</p>
<table>
<thead>
<tr>
<th>Improvement</th>
<th>“Regional” Indicator</th>
<th>Site Specific Indicator</th>
</tr>
</thead>
<tbody>
<tr>
<td>Conspicuous evidence of ground failure</td>
<td>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/significant rotation of retaining structures.</td>
<td>Surface manifestation of liquefaction (e.g., sand boils), obvious evidence of slope movement such as linear fissures in ground parallel to a slope (again, not to be confused with fissuring caused by desiccation of expansive soil), rising or bulging of the ground surface, or collapse/significant rotation of retaining structures.</td>
</tr>
<tr>
<td>Streets, curbs, and sidewalks and site flatwork</td>
<td>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 possible repairs of street following repairs to underground utilities by municipalities.</td>
<td>Fresh-appearing soil cracks or flatwork cracks/separations of construction joints wider than 1/4 inch or fresh-appearing out-of-levelness in patios, decks, etc. not consistent with the normal construction practices (patios adjacent to structures are sloped away from the building for drainage), general level of maintenance, and normal aging. Note that 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.</td>
</tr>
<tr>
<td>Utility line condition</td>
<td>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).</td>
<td>Damage to underground utility lines on the property such as water service lines, sanitary sewer lines, gas lines.</td>
</tr>
<tr>
<td>Swimming pool condition</td>
<td></td>
<td>Fresh-appearing out-of-levelness of the pool coping not consistent with the general level of maintenance and normal aging, and fresh-appearing cracking of the pool shell.</td>
</tr>
<tr>
<td>Foundation condition</td>
<td></td>
<td>Fresh fractures 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 direct onto the exposed foundation edge are indicators of possible ground failure.</td>
</tr>
</tbody>
</table>

4 Crack widths are approximate; background on the selection of these numbers is provided in Chapter 5.
4.3 Surface Fault Rupture

4.3.1 Fault Rupture Mechanisms and Types of Surface Fault Rupture

Fault rupture involves relative displacements (i.e., slip) of blocks of rock on opposite sides of the fault surface. The direction of these relative movements 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 96 illustrates strike-slip and dip-slip faults. Dip-slip faults are represented by the reverse and normal mechanisms, which are defined below.

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 (the block of earth overlying the dipping fault) and the foot wall (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.

For the following discussion, 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. As defined by Youngs et al. (2003), principal faulting is slip along the main plane (or planes) responsible for the release of seismic energy during the earthquake. 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.

Youngs et al. (2003) define distributed faulting as 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. Moreover, as employed herein, 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 sections that follow, 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 97, 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 very irregular alignments, although the shape of the rupture surface is often predictable based on surface topography and geologic mapping. As shown subsequently, distributed faulting from dip-slip earthquakes is more pronounced on the hanging wall than on the foot wall.

4.3.2 Methods of Investigation

An investigation of whether surface fault rupture is a viable mechanism of ground failure for a given site should begin with a review of relevant literature (including web-based publications) 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 Earthquake Engineering Research Institute (EERI) and the U.S. Geological Survey. These types of publications are often available within a few months of the earthquake. The results of such investigations should generally be adequate to evaluate whether principal faulting occurred at or near a site. To evaluate if distributed faulting may have occurred at a site, detailed geologic studies of the site in question and its surrounding area will generally be required.

A site-specific investigation of whether surface fault rupture may have affected a particular site is best performed shortly after the earthquake, while ground deformation features are still evident. Such an investigation should include a thorough reconnaissance of the site in question as well as the surrounding region. The investigator should look for regional ground deformation patterns consistent with surface fault rupture so as to confirm the source of the ground cracking is not another form of ground failure 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 any evidence of ground deformation, then it can be concluded that neither surface rupture nor any other mechanism of seismic-induced ground failure occurred at the site, and therefore could not be responsible for damage to improvements.

If data from site reconnaissance performed shortly after the earthquake is not available, a site-specific investigation of surface fault rupture can at best provide the following information:
1. Proximity of the site to the nearest mapped surface rupture features (from publicly-distributed mapping products, as described above), which can be used to assess the potential for principal faulting at the site.

2. Probability of distributed 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.

The first item above, locations of primary surface rupture, will generally be available in the literature within a few months of a major earthquake. Organizations providing such mapping results were listed previously.

The second item above has been the subject of considerable research for the following conditions: (a) distributed rupture involving discrete breaks across discontinuities for normal fault earthquakes, and (b) distributed rupture involving ground warping for large-magnitude strike-slip earthquakes. Data and models for distributed surface rupture for other conditions are under development by Petersen et al. (2004) but are not complete as of this writing.

The data for distributed rupture associated with normal fault earthquakes was compiled as part of studies for the Yucca Mountain project in Nevada. Figure 98 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 98 to be appropriately applied, and the probabilities obtained only apply to displacements across the discontinuity. From Figure 98, 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 > 10-15 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 99, the probability of distributed rupture decreases as the difference between strikes increases. It should be emphasized that the data in Figure 98 and Figure 99 is 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 has investigated the warping of fences crossing the primary rupture of the San Andreas fault from the 1906 San Francisco earthquake (Wells, personal communication, 2002). The results are shown in Figure 100, and show the variation of ground warping with distance from the primary trace. 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.

4.3.3 Damage, Repair, and Mitigation

In the aftermath of 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?
In this section, a brief overview of repair and mitigation strategies is provided. The conditions under which soil “damage” can occur are also provided. 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.

### 4.3.3.1 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 ground failure 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 in California 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. Accordingly, 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.

### 4.3.3.2 Repair and Mitigation Strategies

Prior to making repair or mitigation recommendations, an engineer 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 future fault rupture at the property. In addition, the engineer must be familiar with their recommended repair/mitigation technique and the technique’s limitations, risks, costs, and appropriateness for the site. Lastly, the engineer must understand their client’s needs and expectations.

The repair/mitigation measures recommended by an engineer must satisfy two criteria. First, the recommendations must be appropriate and consistent with the magnitude of ground deformation and collateral physical damage observed at the property. Second, the recommendations must be consistent with an analysis of the possible fault rupture hazards that may still exist at the property. Appropriate repairs may address only damage to improvements or 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 legally and economically feasible (i.e. in areas of minor surface disruption such as areas of distributed surface rupture), damaged improvements
may be repaired or replaced in kind. Except as discussed in the following subsection, 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 surface fault rupture. Each strategy requires proper interpretation of the geology on both a regional and site-specific basis.

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 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, as such foundations do not allow decoupling of ground displacements from foundation displacements.
4.4 Soil Liquefaction

4.4.1 Liquefaction-Related Phenomena and the Conditions Under Which They Occur

It is has long been recognized that loose, dry, granular soils densify when sheared slowly under drained conditions. If the soils are saturated and shearing occurs rapidly (i.e., undrained conditions), 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.

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 at-grade improvements. Consequences of liquefaction can be grouped into the general categories of flow failure and cyclic mobility. 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. 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. Accordingly, 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.

Unlike some other forms of earthquake-induced ground failure, 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 layer 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) 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 101. Using a larger data set than that of Ishihara (1985), Youd and Garris (1995) found the boundary curves in Figure 101 are accurate for sites not subject to ground oscillation or lateral spread. Criteria for evaluating the potential for lateral spreading are provided in Section 4.4.2.5. Site 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, it should be noted that Figure 101 applies essentially for sandy soils. Its reliability for fine-grained materials has not been verified, and such verification work remains a research need.

When liquefaction effects are manifest at the ground surface, such effects may take the form of landslides (e.g., flow failures), ground fissures, lateral spreads, settlement, and sand boils (e.g., Figure 9 and Figure 10). 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 (e.g., Figure 14). 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 (e.g., Figure 15 and Figure 37). 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 (e.g., Figure 16) or can produce differential settlements. Liquefaction at shallow depths can cause sand boils to form (e.g., Figure 26), which occurs when excess pore pressures in a liquefiable strata vent through cracks or openings in overlying non-liquefied layers.

For structures supported on shallow foundations, liquefaction effects can include foundation bearing failures (e.g., Figure 17), foundation settlement and/or tilting (e.g., Figure 13 and Figure 18), and lateral translations of foundations (e.g., Figure 11, Figure 12 and Figure 19). 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.

4.4.2 Methods of Investigation

4.4.2.1 Site Reconnaissance and Literature Review

An investigation of whether liquefaction is a viable mechanism of ground failure for a given site begins with a thorough reconnaissance of the site as well as the surrounding region. The investigator 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/or foundation settlement relative to surrounding ground.

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. For example, post-earthquake reconnaissance reports by organizations such as the Earthquake Engineering Research Institute (EERI) will typically document locations where liquefaction was and was not observed. However, unless the 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. Other literature that should be reviewed includes liquefaction hazard maps prepared by the California Geological Survey [see Special Publication 118 (1999) and Martin and Lew (1999) and regional compilations of surface geological and groundwater data such as Tinsley et al. (1985)].

If detailed reconnaissance performed shortly after the earthquake does not reveal any 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 and hence did not damage surface improvements. However, if such field reconnaissance data is not available, additional analysis will be required to evaluate the potential for liquefaction. This begins with a screening analysis (described in Section 4.4.2.2). If the screening analysis indicates that the site may be susceptible to liquefaction, more detailed investigations involving subsurface exploration, laboratory testing, and engineering analyses will need to be performed to evaluate whether liquefaction was likely to have been triggered and its effects. These are described in the sub-Sections 4.4.2.3 to 4.4.2.5.

4.4.2.2 Screening Analysis for Liquefaction

The purpose of a screening analysis is to determine whether or not a site has obvious characteristics that would indicate that liquefaction could not have occurred. As discussed in Section 4.1, 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, the guidelines consider a different scenario than what is considered here. The present discussion is based on the assumption that an earthquake has already occurred (i.e., earthquake parameters are well defined and available in literature).

Specific guidelines for assessing liquefaction susceptibility are presented by CDMG (1997) and Martin and Lew (1999) and are adapted here for post earthquake site screening (with some modifications, which are clearly labeled):

- 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.

- 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 California Geological Survey.

- If high fines content soil materials (> 35%) are encountered during site exploration, it is possible that those materials are non-liquefiable on the basis of significant clay content. For purposes of this screening, liquefiable fine-grained soils are those that have both of the following characteristics:
  - Liquid Limit less than 35
  - Water Content greater than 0.9 x Liquid Limit

Conversely, soils that do not meet both of these criteria are likely too clayey to liquefy in the classic sense. However, the absence of liquefaction susceptibility on the basis of these criteria does not necessarily imply that ground failure cannot occur—cyclic softening of saturated clayey soil can still lead to ground failure, especially in the presence of a driving static shear stress (such as from a foundation or slope, see Section 4.5.2.2 for details on shear strength parameter selection for stability analyses in these materials). The above criteria are similar to the so-called Chinese criteria originally presented by Seed and Idriss (1982) and subsequently re-stated in standard references.
such as Martin and Lew (1999) and Youd et al. (2001). The difference between the
criteria listed above and the Chinese criteria is that the Chinese Criteria also have
requirements related to the fraction of the soil with a particle size smaller than 0.005 mm.
These criteria have been found to be ineffective at distinguishing liquefiable and non-
liquefiable soils by Sancio et al. (2002) and Stewart et al. (2003), and hence are not
recommended for use here.

- If the corrected standard penetration blow count, \((N_{I})_{60}\), 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.

### 4.4.2.3 Subsurface Exploration and Laboratory Testing

The objective of subsurface exploration and laboratory testing for liquefaction studies is to
classify the soil stratigraphy, evaluate the depth to groundwater, evaluate soil index properties,
and develop penetration resistance measures for the susceptible soil layers. For sites with
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/SPT or CPT.
*In situ* measurements of soil shear wave velocity \((V_s)\) can be used to supplement, but not generally
not replace, traditional penetration resistance testing. Advantages and limitations of each test are
summarized in Table 4.3. The penetration resistance tests most commonly used in practice are SPT
and CPT.

<table>
<thead>
<tr>
<th>Feature</th>
<th>Test Type</th>
</tr>
</thead>
<tbody>
<tr>
<td><em>In situ</em> tests used to evaluate liquefaction resistance (Youd et al., 2001)</td>
<td></td>
</tr>
<tr>
<td>Feature</td>
<td>SPT</td>
</tr>
<tr>
<td>Past measurements at liquefaction sites</td>
<td>Abundant</td>
</tr>
<tr>
<td>Type of stress-strain behavior influencing test</td>
<td>Partially drained, large strain</td>
</tr>
<tr>
<td>Quality control and repeatability</td>
<td>Poor to good</td>
</tr>
<tr>
<td>Detection of variability of soil deposits</td>
<td>Good for closely spaced tests</td>
</tr>
<tr>
<td>Soil types in which test is recommended</td>
<td>Nongravel</td>
</tr>
<tr>
<td>Test measures index or engineering property</td>
<td>Index</td>
</tr>
</tbody>
</table>

The cone penetration test (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 do not provide samples, and hence CPT alone is not an
appropriate means by which to assess liquefaction susceptibility. 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.
In boreholes, samples should be retrieved by driving a standard penetration test (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 \( (q_c) \) and sleeve friction \( (f_s) \) 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).

### 4.4.2.4 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 in Section 4.4.2.2, more detailed analyses are needed to evaluate whether liquefaction was likely or unlikely to have actually occurred. 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). Several widely-used references that describe the most recent versions of these techniques include Martin and Lew (1999) and Youd et al. (2001, 2003).

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:

\[
FS = \frac{CRR}{CSR} \tag{4.1}
\]

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 suggest that liquefaction was likely to have been triggered, whereas \( FS > 1 \) implies that liquefaction was unlikely. 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 4.1 is the subject of the following sub-sections.

**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 4.4.2.3); (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 Martin and Lew (1999) and Youd et al. (2001). 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*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. These corrections are provided in Table 7.2 of Martin and Lew (1999). The resulting penetration resistance values for SPT and CPT are referred to as (*N*₆₀)₆₀ and *q*ₑ₁₅₆₀, 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 *M* = 7.5 earthquakes and shallow depths (corresponding to vertical effective stress *σ*’ᵥ < 100 kPa) can be evaluated based on diagrams presented by Martin and Lew (1999) and Youd et al. (2001), and reproduced in Figure 102. Because of the magnitude and overburden pressure constraints on the CRR parameter retrieved from Figure 102, it is referred to here as CRR*.

The correction for magnitude is necessary because the number of cycles of earthquake shaking is strongly dependent on earthquake shaking, 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 *C*ₘ. This parameter is provided by Martin and Lew (1999) and Youd et al. (2001), and is shown here in Figure 103.

The correction for overburden stress is necessary because (1) the case histories of liquefaction and non-liquefaction that populate the data sets shown in Figure 102 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*ₛ. This parameter is provided by Martin and Lew (1999) and Youd et al. (2001), and is shown in Figure 104.

After evaluation of correction factors *C*ₘ and *K*₄, the value of CRR for use in Equation 4.1 can then be evaluated as follows:

\[
CRR = CRR^* C_M K_\sigma
\]  

(4.2)
It should be noted that several recent developments in liquefaction resistance evaluations have occurred that are not reflected by the CRR evaluation procedures discussed above. These are briefly identified below:

1. New procedures for evaluation of CRR* have been presented for both SPT and CPT data by Idriss and Boulanger (2004) and Seed et al. (2003). These procedures are based on larger and more carefully screened databases than those reflected in Figure 102. In addition, the Seed et al. (2003) procedures present curves for probability of liquefaction instead of single lines separating the liquefaction and non-liquefaction spaces.

2. New magnitude scaling factors have been proposed by Liu et al. (2001) that are distance-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. New C_M values that are dependent only on magnitude have also been presented by Seed et al. (2001) and Idriss (1999), and 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).

3. New overburden correction factors have been proposed by Boulanger (2003) and Seed et al. (2001, 2003) that vary continuously with \( \sigma'_v \) (instead of being one for \( \sigma'_v < 100 \) kPa and decreasing for \( \sigma'_v > 100 \) kPa).

In the opinion of the authors, each of the above developments represents improvements over the aforementioned simplified procedure. However, these changes have not found their way into routine geotechnical practice as of this writing, which is why they are not yet incorporated into the widely accepted simplified procedure of Youd et al. (2001, 2003). Potential users of the new procedures should be cautioned that such procedures must be used in their entirety, meaning that components of the new and old procedures should not be combined (e.g., CRR* from Figure 102 should not be corrected using \( K_\sigma \) values from Seed et al., 2001).

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).

**Cyclic Stress Ratio (CSR)**

The cyclic stress ratio is calculated by the following formula originally developed by Seed and Idriss (1971)

\[
CSR = 0.65 \left( \frac{a_{max}}{g} \right) \left( \frac{\sigma_{vo}}{\sigma_{vo}} \right) r_d
\]  

(4.3)

where
$a_{\text{max}} = \text{peak horizontal acceleration at ground surface}$

g = \text{acceleration of gravity}$

$\sigma_{v0}' = \text{effective vertical overburden stress}$

$\sigma_{v0} = \text{total vertical overburden stress}$

$r_d = \text{stress reduction factor}$

Guidance for the evaluation of $a_{\text{max}}$ are provided in Chapter 3. Youd et al. (2001) provide guidance for selection of stress reduction coefficient, $r_d$. It should be noted that new procedures for evaluation of $r_d$ have been presented by Idriss and Boulanger (2004) and Seed et al. (2001, 2003) that are appropriate for use with their respective liquefaction triggering analysis methods.

### 4.4.2.5 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 sub-divides the broad subject of liquefaction consequence into sub-sections on stability problems, lateral spreading, and post-liquefaction re-consolidation settlement.

#### Stability Problems (Undrained Residual Strength)

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, retaining walls, etc. 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 $S_r/\sigma_{v0}'$ 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 attached as Figure 105.

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/design. While much work remains to be done on this issue, the authors’ recommendation at present is to give 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 occur with the Stark and Olsen procedure but could occur 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.
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 is covered in Sections 4.5.2.4 and 4.7, respectively. Analysis procedures for foundation displacement under such conditions are not currently available. Development of such procedures is a research need.

**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 ground failure that, in a post earthquake investigation, shifts the question of whether or not ground failure occurred to what is the appropriate repair and mitigation strategy. However, in some instances, it may be necessary to differentiate between ground failure modes or to estimate the magnitude of displacement from lateral spreading. This is generally accomplished using the empirical approach described below. It should be noted that this approach is valid for ground slopes between 0.1-6% and for distances behind a free face ranging from 5-100 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 previous sub-section).

Lateral spread displacements can be estimated using an empirical model based on multilinear regression of a large case history database. The most recent and recommended version of this approach is described in Youd et al. (2002). The method is applicable to sites with gently sloping ground or ground with a free face (i.e., drainage or stream channels or bodies of water) and has been widely used in engineering practice. The first step in applying this method is to determine whether or not liquefaction has been triggered at the site according to the procedures described in Section 4.4.2.4. Provided liquefaction was triggered, the Youd et al. (2002) manuscript should be consulted for guidance regarding calculations of lateral spread displacements.

**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).

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 values to correct for the presence of fines. These correction factors are generally taken as the same values used for triggering analysis and discussed in Section 4.4.2.4. When clayey sands are present and satisfy the modified “Chinese criteria” listed in Section 4.4.2.2, 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 101). Care should be employed when using Figure 101 as 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 certain conditions (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.
4.4.3 Damage, Repair, and Mitigation

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?

In this section, a brief overview of repair and mitigation strategies is provided. The conditions under which soil “damage” can occur are also provided. 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.

4.4.3.1 Soil Damage

The effect of liquefaction and post-liquefaction re-consolidation 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.

4.4.3.2 Repair and Mitigation Strategies

Prior to making repair or mitigation recommendations, an engineer 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. In addition, the engineer must be familiar with their recommended repair/mitigation technique and the technique’s limitations, risks, costs, and appropriateness for the site. Lastly, the engineer must understand their client’s needs and expectations.

The repair/mitigation measures recommended by an engineer must satisfy two criteria. First, the recommendations must be appropriate and consistent with the magnitude of ground deformation and collateral physical damage observed at the property. Second, the recommendations must be consistent with an analysis (as described above and elaborated in more detail in CDMG, 1997) of the possible deformations and liquefaction hazards that may still exist at the property. Appropriate repairs may address only damage to improvements or 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 or replace the damaged improvements in-kind and accept the risk of possible liquefaction in future events. Such repair strategies may include re-leveling of the structure on the foundation, re-leveling of the foundation using available techniques (e.g., pressure grouting), or re-leveling and repair of fractured foundations.

In cases of severe liquefaction or where mitigation of the liquefaction hazard is desired, structure modification and/or ground improvement should be considered. CDMG (1997) and Martin and Lew (1999) contain a detailed discussion of these options along with references.

Structure 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 nature 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 may be feasible. Under other conditions, construction of new foundations such as post tensioned slabs, mat foundations, piles or caissons may be 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 must be verified/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, or 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 repair/mitigation of a site. For example, an appropriate combination of ground improvement methods may be used at a site or a combination of ground improvement and structural modification may also be possible.
4.5 Seismically-induced Landslides

4.5.1 Types of Landslides and Conditions Under Which They Occur

A significant source of ground failure during earthquakes is landslides, defined for the purpose of this discussion as seismically-induced permanent shear deformations within geologic media. These shear deformations need to 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 Sections 4.4 and 4.6.

Whether induced by earthquakes or other processes, landslides can be subdivided into several generalized categories (Varnes, 1978; Keefer, 1984):

1. Masses of disrupted slide material, such as rock falls or avalanches.
2. Relatively coherent slide masses whose displacement is accommodated along well-defined slip surfaces or across relatively broad, distributed shear zones.
3. Lateral spreads and flows associated with soil strength loss due to pore pressure increase.

Examples of these types of landslides are shown in Figure 32 to Figure 37. 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 is a crucial step in engineering analyses of slope stability, because different analysis procedures are appropriate for different landslide mechanisms.

As described by Keefer (1984) and illustrated in Figure 32, disrupted slides and falls occur in areas of high topographic relief (slopes steeper than 35-40 degrees) and tend to involve closely jointed or weakly cemented materials. 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 travel large distances (on the order of kilometers) 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 further below).

Coherent slides can occur in rock or soil materials and at slope angles much lower than those for disrupted slides and falls. 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 (e.g., Figure 36). 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 (sliding occurring within the embankment materials or in relatively soft foundation soils, e.g. 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 Section 4.4. 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 shear stresses, flow slides will occur that can involve very large displacements, such as shown in Figure 37.

Ridgetop fissuring and shattering is a result of amplification or focusing of seismic energy due to local topographic effects. This phenomena 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 extensial 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).

Sections 4.3.1 and 4.4.2.1 discussed identification of the fault rupture and lateral spreading, the following section discusses identification of landsliding.

4.5.2 Methods of Investigation

4.5.2.1 Reconnaissance and Geologic Investigation

An investigation of whether seismically-induced landsliding is a viable mechanism of ground failure for a given site begins with a thorough reconnaissance of the site in question as well as the surrounding region. The investigator 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. 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 re-development process). Bedding planes that intersect a slope or which are sub-parallel 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. Any 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 any evidence of ground deformation, it is not likely that landslide phenomena occurred at the site. However, if such field reconnaissance data is not available, detailed analytical studies will be needed to investigate the potential for landslide development during the earthquake. Those studies entail subsurface exploration, laboratory testing, and engineering analyses. Those tasks are described in the following sub-sections.

4.5.2.2 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 (Modified California sampler). Guidelines on subsurface exploration and sampling techniques are provided in Chapters 4 and 6 of a document for slope stability evaluations (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 (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 the Blake et al. (2002) slope stability document, 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:

1. 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, seismically-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 (< 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.

2. 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).

3. 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 below in Section 4.5.2.4. 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.
4. 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% to 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.

5. 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).

4.5.2.3 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 state-of-the-art 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 report 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 106. This assumption implies a uniform factor of safety ($FS$) across the slide surface, where $FS$ is defined as:

$$ FS = \frac{\text{Available Shear Strength}}{\text{Equilibrium Shear Stress}} $$

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 input parameters from site-to-site. 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 factor of safety ($FS$) 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 4.4 on page 44 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 methods of Morgenstern and Price (1965), Spencer (1967), Sarma (1973), Taylor (1948), and Janbu's (1968) generalized procedure of slices 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) has found all of these methods to provide answers within 5% of each other.

### 4.5.2.4 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 Section 4.4. 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 in Section 4.4. 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/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 sub-sections.

**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 107. 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 (e.g., 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 108. 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 $0.65 \times k_{max}$.

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 likely to have not 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.
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 109, 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.

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 \( k_y = \frac{a_y}{g} \), 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 in Section 4.5.2.2.

As illustrated in Figure 110, 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, the differential velocity begins at the instant of time when acceleration first exceeds \( a_y \) (Point A in Figure 110) and increases throughout the time period during which \( a > a_y \). When the acceleration drops below \( a_y \) (Point B in Figure 110), the differential velocity is at a local maximum. Differential velocity will decrease while \( a < a_y \) until it goes to zero (Point C in Figure 110), 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 110), the differential displacement is simply calculated by integrating across the differential velocity time history (as shown in the bottom frame of Figure 110).

The above procedure is convenient to apply, especially with the availability of modern computer programs that can efficiently perform calculations for many time histories (e.g., 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 engineer, 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.

- 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 in Section 4.5.2.2.

- 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 $k_{max} = MHEA/g$. 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 maturely developed procedure is that of Bray and Rathje (1998) and Bray et al. (1998). This procedure was originally developed for landfills but has also found recent widespread use for hillside residential and commercial construction (Blake et al., 2002). The procedure has two basic steps: analysis of the seismic demand accounting for vertical incoherence effects and 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 from calculations of wave propagation through equivalent one-dimensional slide masses. The results of these calculations are shown in Figure 111, where MHEA is normalized by the product of MHA 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 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×NRF) differs from one as a result of vertical ground motion incoherence within the slide mass and is related in Figure 111 to the ratio of the small-strain period of the sliding mass (T_s) to the mean period of the input motion (T_m). The ratio MHEA/(MHA×NRF) is less than one for T_s/T_m ∼ 0.5 and is variable with an average of about 1.0 for T_s/T_m < ∼ 0.5.

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 = MHEA/g), significant duration of shaking (measured as the time between 5-95% normalized Arias intensity, D_{5-95}) 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 magnitude 6.25 to 8 earthquakes at each of four k/y/k_max ratios. The model and data are shown in Figure 112 and indicate a lognormal distribution of normalized displacement u/(k_max D_{5-95}) for a given k/y/k_max ratio. The median of this lognormal distribution is described by,

$$\log_{10}\left(\frac{u}{k_{\text{max}} \cdot D_{5-95}}\right) = 1.87 - 3.477 \cdot \frac{k_y}{k_{\text{max}}}$$

(4.5)

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 calculated displacement are zero with reasonable assumptions of ground motion and soil strength characteristics, then seismically-induced slope displacement 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 seismically-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 (e.g., Blake et al., 2002) suggest that calculated Newmark displacements < 5 cm in occupied structures and < 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.

4.5.3 Damage, Repair, and Mitigation

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?

In this section, a brief overview of repair and mitigation strategies is provided. The conditions under which soil “damage” can occur are also provided. In this context, soil damage is defined as an earthquake-induced disruption of the soil that weakens the soil, causing landsliding in future events to be more likely than it had been prior to the earthquake.

4.5.3.1 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, it is possible that the slope materials 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 is only possible in relatively clayey soil or clayey rock materials and occurs because of clay fabric re-orientation such that clay particles are aligned with the direction of slip. This phenomenon would not be expected in nonplastic sands, silts, gravels, or bedrock comprised of those material types because of the lack of platy particle shapes. Another circumstance under which soil damage could be envisioned 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 to be soil damage.

In either of the above cases, 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.

If a site has no evidence of earthquake induced ground deformation, or if deformations occurred under conditions different from those describe above (e.g., raveling of a fill slope or re-activation of
an pre-existing landslide), the slope material’s resistance to landsliding would not be expected to have been compromised as a result of the earthquake.

4.5.3.2 Repair and Mitigation Strategies

Prior to making repair or mitigation recommendations, an engineer 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. In addition, the engineer must be familiar with their recommended repair/mitigation technique and the technique’s limitations, risks, costs, and appropriateness for the site. Lastly, the engineer must understand their client’s needs and expectations.

The repair/mitigation measures recommended by an engineer must satisfy two criteria. First, the recommendations must be appropriate and consistent with the magnitude of ground movement and collateral physical damage observed at the property. Second, the recommendations must be consistent with the potential for ongoing or future movement of the slide mass. Appropriate repairs may address only damage to improvements or 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 to (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 inadequate to mitigate the landslide hazard, it may be necessary to abandon the site.

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 4.4 for details).

Dewatering can be a very effective mitigation strategy, but the de-watering system must 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.
The resistance of structures to slope deformations can be enhanced through reinforcement of foundations or modifying the foundation configuration. For example, drilled shafts and grade beams can be installed in lieu of shallow foundations such as footings. 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).
Table 4.4 Characteristics of commonly used methods of limit equilibrium analysis
(after Duncan, 1996)

<table>
<thead>
<tr>
<th>Method</th>
<th>Equilibrium Conditions Satisfied</th>
<th>Shape of Slip Surface</th>
<th>Assumptions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Friction Circle Method (Taylor, 1948)</td>
<td>Moment and force equilibrium</td>
<td>Circular</td>
<td>Resultant tangent to friction circle</td>
</tr>
<tr>
<td>Ordinary Method of Slices (Fellenius, 1927)</td>
<td>Moment equilibrium of entire mass</td>
<td>Circular</td>
<td>Normal force on base of slice is $W \cos \alpha$ and shear force is $W \sin \alpha$</td>
</tr>
<tr>
<td>Bishop's Modified Method (Bishop, 1955)</td>
<td>Vertical equilibrium and overall moment equilibrium</td>
<td>Circular</td>
<td>Side forces are horizontal</td>
</tr>
<tr>
<td>Janbu's (1968) Simplified</td>
<td>Force equilibrium</td>
<td>Any shape</td>
<td>Side forces are horizontal</td>
</tr>
<tr>
<td>Lowe and Karafiath's (1960) Method</td>
<td>Vertical and horizontal force equilibrium</td>
<td>Any shape</td>
<td>Side force inclinations are average of slope surface and slip surface (varies from slice to slice)</td>
</tr>
<tr>
<td>Janbu's (1968) Generalized Method</td>
<td>All conditions of equilibrium</td>
<td>Any shape</td>
<td>Assumes heights of side forces above the base vary from slice to slice</td>
</tr>
<tr>
<td>Spencer's (1967) Method</td>
<td>All conditions of equilibrium</td>
<td>Any shape</td>
<td>Inclinations of side forces are the same for every slice; side force inclination is calculated in the process of the solution</td>
</tr>
<tr>
<td>Morgenstern and Price's (1965) Method</td>
<td>All conditions of equilibrium</td>
<td>Any shape</td>
<td>Inclinations of side forces follow a prescribed pattern; side forces can vary from slice to slice</td>
</tr>
<tr>
<td>Sarma's (1973) Method</td>
<td>All conditions of equilibrium</td>
<td>Any shape</td>
<td>Magnitudes of vertical side forces follow prescribed patterns</td>
</tr>
</tbody>
</table>
4.6 Seismic Compression

4.6.1 Definition and Requisite Conditions

Seismic compression is defined as the accrual of contractive volumetric strains in unsaturated soil during strong shaking from earthquakes. Ground deformations in compacted fill slopes from seismic compression have been well documented in the literature (e.g., 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 (e.g., SSC, 1995).

Stewart et al. (2001) documented a large number of case histories of ground deformations in fill from the 1994 Northridge Earthquake. Characteristic fill deformation features are illustrated in Figure 52 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 simply 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. In the following section, analysis procedures intended strictly for fill soils are distinguished from those that can also be used for natural soils.

4.6.2 Methods of Investigation

4.6.2.1 Reconnaissance

An investigation of whether seismic compression is a viable mechanism of ground failure for a given site begins with a thorough site reconnaissance. Unlike fault rupture and landslides, seismic compression tends to be a localized phenomena, and hence the reconnaissance activities should be focused on the site in question and the immediately surround area. The investigator should look for local ground deformation patterns such as outlined above (e.g., differential settlement and/or extension of fill pads). 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 photo review, or as-built grading plans for the site. Ground deformations should be interpreted relative to the fill section geometry to evaluate whether there is any 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 any 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 is not available, detailed investigations will be necessary to investigate the likelihood that seismic compression occurred at the site. These investigations involve subsurface exploration, laboratory testing, and engineering analyses. Those phases of the investigation are described in the following sub-sections.
4.6.2.2 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 (i.e., 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 (e.g., 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)

The above 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. Relative compaction is measured using the Modified Proctor standard (ASTM D 1557). For clean sands, it is nearly impossible to obtain undisturbed samples, and so 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 Maine (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 (Section 4.4).
4.6.2.3 Analysis of 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 (PI > approximately 30) tend to have a low susceptibility to seismic compression.

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/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 (natural soil or fill). The second procedure was developed as part of research funded by the CUREE Earthquake Damage Assessment (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 plasticities. An early version of the compacted fills procedure has been verified relative to three well-documented case histories by Stewart and Whang (2003).

Tokimatsu and Seed (1987) Procedure for Clean Sands

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:

\[
\tau_{\text{rigid, max}} = \frac{PHA}{g} \cdot \sigma_0
\]  

where \( g \) = the acceleration due to gravity and \( \sigma_0 \) = total overburden pressure at depth \( h \).

Soil flexibility reduces the shear stress to values less than \( \tau_{\text{rigid, max}} \) 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 \( \tau_{\text{rigid, max}} \) by a stress reduction factor, \( r_d \) (which is the ratio of the actual shear stress at depth vs. the theoretical “rigid body” shear stress). A factor of 0.65 is then applied to reduce the peak cyclic shear stress, \( \tau_{\text{max}} \), to the effective cyclic stress, \( \tau_{\text{eff}} \), as:

\[
\tau_{\text{eff}} = 0.65 \cdot \frac{PHA}{g} \cdot \sigma_0 \cdot r_d
\]  

Effective shear strain, \( \gamma_{\text{eff}} \), is estimated from \( \tau_{\text{eff}} \) using the effective shear modulus \( (G_{\text{eff}}) \), as follows:

\[
\gamma_{\text{eff}} = \frac{\tau_{\text{eff}}}{G_{\text{eff}}} = \frac{\tau_{\text{eff}}}{G_{\text{max}} \left( \frac{G_{\text{eff}}}{G_{\text{max}}} \right)}
\]  

47
where \( G_{\text{max}} \) = small strain shear modulus. Combining Equations 4.7 and 4.8 leads to:

\[
\gamma_{\text{eff}} \frac{G_{\text{eff}}}{G_{\text{max}}} = 0.65 \cdot PHA \cdot \frac{\sigma_0 \cdot r_d}{g \cdot G_{\text{max}}} \equiv P \tag{4.9}
\]

The product \( \gamma_{\text{eff}} \left( G_{\text{eff}}/G_{\text{max}} \right) \) in Equation 4.9 can be readily translated to a shear strain amplitude \( \gamma_{\text{eff}} \) using published models for soil modulus reduction with increasing shear strain (i.e. models relating \( \gamma_{\text{eff}} \) to \( G_{\text{eff}}/G_{\text{max}} \)). Tokimatsu and Seed (1987) recommended using the modulus reduction curves of Iwasaki et al. (1978), which depend on effective stress.

Having estimated \( \gamma_{\text{eff}} \) 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 \( \gamma_{\text{eff}} \) 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 \( \varepsilon_v \) 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 \( \varepsilon_v \) is multiplied by two to account for multi-directional shaking effects per the recommendations of Pyke et al. (1975). Hence, the final estimate of volumetric strain at a point is represented by \( 2 \times C_N \times (\varepsilon_v)_{N=15} \). These volumetric strains are then integrated over the depth of the soil column to calculate settlement.

**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 4.9, with stress reduction factor \( r_d \) evaluated based on the model of Seed et al. (2001) as follows:

\[
z < 20 \text{ m: } r_d = \frac{\left[1 + a_1/a_2(z)\right]}{\left[1 + a_1/a_3\right]} \tag{4.10}
\]
\[ z > 20 \text{ m}: r_d = \frac{[1 + a_1/a_2(z = 20)]}{[1 + a_1/a_3]} - 0.0046(z - 20) \]  
\quad (4.11)

where,

\[ a_1 = -23.013 - 2.949 \cdot PHA / g + 0.999 \cdot m + 0.0053 \cdot V_{s-12} \]  
\quad (4.12)

\[ a_2(z) = 16.258 + 0.201 \cdot e^{0.341[-z + 0.0785V_{s-12} + 7.586]} \]  
\quad (4.13)

\[ a_2(z=20) \text{ is } a_2(z) \text{ with } z \text{ set to } 20 \text{ m} \]  
\quad (4.14)

\[ a_3 = 16.258 + 0.201 \cdot e^{0.341(0.0785V_{s-12} + 7.586)} \]  
\quad (4.15)

\[ z = \text{depth in meters} \]

\[ m = \text{earthquake magnitude} \]

\[ V_{s-12} = \text{average shear wave velocity in upper 12 m of the site (in m/s)} \]

3. Estimate the equivalent number of uniform stain cycles \(N\) based on earthquake magnitude and site-source distance using Figure 113 or the following:

\[
N = \left( \frac{\exp\left(b_1 + b_2(m - m^*)\right)}{10^{1.5m+16.05}} \right)^{1/3} + Sc_1 + rc_2
\]  
\quad (4.16)

where \(r\) is in km, \(b_1 = 1.53, \ b_2 = 1.51, \ c_1 = 0.75, \ c_2 = 0.095, \ \beta = 3.2, \ \text{and } m^* = 5.8.\) Parameter \(S\) is zero if rock or shallow soil (< 20 m) underlies the fill and one if > 20 m soil underlies the fill.

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 (\(\sigma'\)), soil plasticity (as represented by plasticity index, PI), and overconsolidation ratio (OCR). Figure 114 shows a family of modulus reduction curves (based on the D&S model) for varying PI and \(\sigma'\) (the effects of OCR are generally small, and the plots in Figure 114 apply for OCR = 1, which is generally appropriate for fills at \(z > 3-6\) m, Duncan et al., 1991). Note that the plots in Figure 114 are formatted to directly estimate shear strain, \(\gamma\) from the product \(\gamma'(G/G_{\text{max}})\).

Pradel (1998) developed a fit to the Iwasaki et al. (1978) curves shown in Figure 114 using the following equation:

\[
\gamma = \frac{1 + g_1 \cdot e^{g_2 \cdot P}}{1 + g_1} \cdot P \cdot 100 \text{ (in %)}
\]  
\quad (4.17)
where $P$ is the product computed in Equation 4.9. The same regression equation is used for the D&S curves, with $g_1$ and $g_2$ related to soil type as follows:

$$PI \approx 30: \quad g_1 = 4.0 \quad g_2 = 1400$$  \hspace{1cm} (4.18)

$$PI \approx 15: \quad g_1 = 0.194 \cdot (\sigma'/p_a)^{0.265} \quad g_2 = 7490 \cdot (\sigma'/p_a)^{-0.418}$$  \hspace{1cm} (4.19)

$$PI \approx 0: \quad g_1 = 0.199 \cdot (\sigma'/p_a)^{0.231} \quad g_2 = 10850 \cdot (\sigma'/p_a)^{-0.410}$$  \hspace{1cm} (4.20)

where $p_a = 101.3$ kPa. Shear strains for intermediate PIs can be interpolated.

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, $\gamma_c$, 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. Volumetric strains obtained by these procedures are multiplied by two to account for multi-directional shaking effects.

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).

**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, $\gamma_c$, 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). 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).

**Clean Sands:** A series of 14 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 $\gamma_c = 0.1$ to 1%. The relationship between $\gamma_c$ and vertical strain at 15 cycles ($\varepsilon_v$) from the tests are shown in Figure 115 for $D_r = 60\%$. Also shown in Figure 115 is a power-law curve fit through the data, ± one standard deviations on the fit, and a line fit through the Silver and Seed (1971) data. The power law fit is described by the following equation:

$$\gamma_c > \gamma_{0v}; (\varepsilon_v)_{N=15} = a \cdot (\gamma_c - \gamma_{0v})^b$$

$$\gamma_c < \gamma_{0v}; (\varepsilon_v)_{N=15} = 0$$

where $a$, $b$, and $\gamma_{0v}$ are dependent on soil composition and compaction condition, and are estimated from the laboratory testing. No trends in the $(\varepsilon_v)_{N=15}$-$\gamma_c$ 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 $\gamma_c$. Residuals of the power-law fit are approximately normally distributed, hence the variability is characterized by a coefficient of variation of COV = 0.37. Note that the median fit in Figure 115 is generally consistent with the Silver and Seed (1971) results.

Figure 116 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 115. 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 115 and Figure 116). The median curves in Figure 116, or the corresponding regression coefficients shown in the figure for use with Equation 4.21, can be used to estimate $(\varepsilon_v)_{N=15}$ for clean sands. Note that in Figure 116 parameter $\gamma_{0v}$ 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 4.21 and the coefficients in Figure 116 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:

$$C_N = R \cdot \ln(N) + c$$

All soils must have $C_N = 1$ at $N=15$, which implies that intercept parameter $c = 1-\ln(15)\times R$. Consequently, the $C_N-N$ relationship for a given soil is fully described by slope parameter $R$. Values of $R$ for sands are shown in Figure 117. Parametric studies indicate that $R$ does not vary with $\gamma_c$ 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.

Soils with Non-Plastic Fines: A series of 8 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 nonplastic (i.e., unmeasurable plastic limit). Simple shear tests were performed using the same protocols as for the sands.
The $\gamma_c \cdot (\varepsilon_{v})_{N=15}$ test data from soil mixtures with 50% sand and 50% silt by weight are shown in Figure 118. These materials have unmeasurable liquid limit based on ASTM procedures, but LL is estimated as $< \sim 17$. 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 118. 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 116 and Figure 118.

The $CN-N$ data from the CSS tests are nearly log-linear as described by Equation 4.22. As with the data for sands, parameter $R$ was found to be independent of other parameters (e.g., as shown in Figure 119). The median value of $R$ for nonplastic, silty sands is 0.36 with a standard deviation of 0.04.

Note by comparing Figure 115, Figure 116 and Figure 118 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 118 and Figure 119 is likely limited.

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 varying from PI = 2 – 27. The results enable the development of volumetric strain material models for soils with variable plasticity.

As shown in Figure 120, 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 $CN-N$ relationship, is effectively independent of other parameters (Figure 121), and has a mean value of about 0.32.

As shown in Figure 122, 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 $\sim 80\%$, 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 123), and has a mean value of about 0.34.
As shown in Figure 124, high plasticity materials (tested material has PI = 27, LL = 47) at low $RC \approx 87\%$ exhibit $S$-dependent behavior similar to those at intermediate plasticity. No effect of saturation was observed at higher $RC \approx 92\%$, as the clod structure was largely broken down during compaction at those relatively high densities. Interestingly, no measurable effect of density (between $RC \approx 87\%-92\%$) 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 122. Parameter $R$ from the test data is shown in Figure 125, and has a mean value of 0.25 and standard deviation 0.04.

### 4.6.3 Damage, Repair and Mitigation

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 section, a brief overview of repair and mitigation strategies is provided. The conditions under which soil “damage” can occur are also provided. 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.

#### 4.6.3.1 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.

#### 4.6.3.2 Repair and Mitigation Strategies

Prior to making repair or mitigation recommendations, an engineer 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 “loose” soils at the property, and the potential future volumetric strain-induced movement at the property. In addition, the engineer must be familiar with their recommended repair/mitigation technique and the technique’s limitations, risks, costs, and appropriateness for the site. Lastly, the engineer must understand their client’s needs and expectations.

The repair/mitigation measures recommended by an engineer must satisfy two criteria. First, the recommendations must be appropriate and consistent with the magnitude of ground deformation and collateral physical damage observed at the property. Second, the recommendations must be consistent with an analysis of the deformation potential that may still exist at the property. Appropriate repairs may address only damage to improvements or include mitigation measures as well, depending upon circumstances.
Ground displacements from seismic compression damage structures by inducing differential settlements and possible local ground extension. Repair strategies may include re-leveling of the structure on the foundation, re-leveling, repair, and/or underpinning of the foundation using available techniques, or the installation of replacement foundations that may include relatively strong and stiff shallow foundations or deep foundations such as drilled shafts inter-connected by grade beams.
4.7 Failures of Retaining Walls Caused by Earthquakes

A retaining wall is defined as any wall that retains soil or rock to maintain a change in elevation, ASCE (1994). 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, etc.) without excessive deformation. In service, most retaining walls deform to some degree. When retaining wall deformation, whether seismically-induced or otherwise, becomes excessive, the retaining wall is said to have “failed.” However, with the exception of obvious collapse or imminent collapse, the magnitude of retaining wall deformation that constitutes failure, or even damage, has not been well defined.

During earthquake shaking, cantilever and gravity retaining walls may move by sliding and/or tilting (Figure 44); reinforced soil slopes/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 (Figure 45). 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/walls, slope inclination and reinforcement stiffness and spacing (Nova-Roessig, 1999 and 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 and 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 otherwise. The focus of this section is on distinguishing between conditions that may have been seismically-induced and those conditions resulting from long-term processes and assessing whether the seismically-induced conditions have damaged or caused failure of the wall. Retaining wall damage is defined as conditions that reduce the wall’s stability below minimum requirements under reasonable future loading conditions, materially alter its serviceability, or 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.

Seismically-induced retaining wall deformations may result from either ground failure or loading of the wall. Seismically-induced ground failure may cause deformations (settlement, lateral movement, global instability) in the supporting ground that produce retaining wall movement independently of the earthquake loading on the wall. Investigations of these modes of ground failure are discussed in preceding sections. The focus of this section are deformations of residential retaining structures (e.g., Figure 43) resulting solely from seismically-induced loading of the wall that may have altered the stability or serviceability of the wall.
4.7.1 Methods of Investigation

4.7.1.1 Site Reconnaissance

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 investigator 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 ground/flatwork cracking behind the retaining wall that may suggest either wall movement or ground failure (e.g., Figure 46 to Figure 49, Figure 50, Figure 54, Figure 56 and Figure 57). Any observed wall movement and distress should be carefully examined to distinguish between seismically-induced wall movement and long-term wall movement (e.g., Figure 80). Note that narrow (i.e. < 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 any evidence of ground deformation, seismically-induced wall movement or fresh appearing cracking of the wall, it is likely that seismically-induced retaining wall damage did not occur at the site, or seismically-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 possible occurrence of structural damage and wall rotation. Investigators should measure the wall rotation at regularly spaced intervals. For walls where the rotation exceeds levels required to mobilize active and passive earth pressures (e.g., Figure 126), the investigator 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 will need to be performed when field indicators of possible seismically-induced wall movement are present (e.g., rotation in excess of that required to mobilize active and passive earth pressures (e.g., Figure 126), unusual cracking of the wall is observed, and/or distress to the ground/flatwork behind 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 may also be required to help distinguish between wall damage caused by long-term static processes and seismic loading. The investigation techniques are described in the following sub-sections.

4.7.1.2 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 (i.e., 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.

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 Section 4.6.2.2. 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 Section 4.5.2.2.

Examination of the drainage system and cantilever retaining wall stem (discussed in the previous section) may be performed within a test pit excavated behind the face of the wall at the location of maximum wall rotation.

### 4.7.1.3 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 shown in Figure 106.

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 and in ASCE (1994). 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:

$$FS = \frac{\text{Available Resisting Forces or Moments}}{\text{Driving Forces or Moments}}$$

(4.23)

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.
(Figure 126). A retaining wall has reserve capacity when \( FS > 1 \). Table 4.5 on page 66 contains a summary of the typical factors of safety utilized for retaining wall design.

### 4.7.1.4 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 and Mononobe and Matsuo, 1929). 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 in Section 4.5.2.4 (Richards and Elms, 1979, Elms and Richards, 1990, and Whitman and Liao, 1985). For reinforced soil slopes/walls, a deformation-based analysis procedure has been proposed by Nova-Roessig (1999).

#### 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 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. The Mononobe-Okabe analysis is described in more detail in Seed and Whitman (1970), Whitman and Liao (1985), and ASCE (1994).

Figure 127 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. The total driving (active) and resisting (passive) forces on a wall is expressed as:

\[
P_{AE} = P_A + \Delta P_{AE} = \frac{1}{2} K_{AE} \gamma H^2 (1 - k_v) \quad (4.24)
\]

\[
P_{PE} = P_P + \Delta P_{PE} = \frac{1}{2} K_{PE} \gamma H^2 (1 - k_v) \quad (4.25)
\]

where

\[
K_{AE} = \frac{\cos^2 (\phi - \Psi - \theta)}{\cos \Psi \cos^2 \theta \cos (\Psi - \theta + \delta) \left[ 1 - \frac{\sin (\phi + \delta) \sin (\phi - \Psi + \beta)}{\cos (\beta - \theta) \cos (\Psi - \theta + \delta)} \right]^2} \quad (4.26)
\]

\[
K_{PE} = \frac{\cos^2 (\phi - \Psi + \theta)}{\cos \Psi \cos^2 \theta \cos (\Psi - \theta + \delta) \left[ 1 - \frac{\sin (\phi + \delta) \sin (\phi - \Psi + \beta)}{\cos (\beta - \theta) \cos (\Psi - \theta + \delta)} \right]^2} \quad (4.27)
\]
where

\[ P_A, P_P = \text{static active and passive forces, respectively} \]
\[ \Delta P_{AE}, \Delta P_{PE} = \text{dynamic active and passive forces, respectively} \]
\[ P_{AE}, P_{PE} = \text{sum of static and dynamic active and passive forces, respectively} \]
\[ \gamma = \text{unit weight of soil} \]
\[ \phi = \text{friction angle of soil} \]
\[ k_v = \text{vertical seismic coefficient} \]
\[ k_h = \text{horizontal peak seismic coefficient} \]
\[ \psi = \tan^{-1}\left(\frac{k_h}{1-k_v}\right) \]
\[ H = \text{height of wall} \]
\[ \theta = \text{inclination of wall with respect to vertical} \]
\[ \delta = \text{wall friction angle} \]
\[ \beta = \text{inclination of backfill behind wall} \]

The quantity \( k_h^5 \) 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 \( g \)). 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/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 Eq. 4.24 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/resonance effects, but not amplification due to impedance contrasts. Shown in Figure 131 is 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 (\( H/\lambda \to 0 \)), the distance from the top of the backfill to the dynamic resultant \( (h_d) \) goes to 0.33H, as noted above. This condition persists to \( H/\lambda \approx 1/8 \), which corresponds to \( f = V_s/8H \). Accordingly, wave reversal/resonance effects are negligible for components of ground motion with frequencies less than half the fundamental frequency of the

---

5 The quantity \( k_h \) as used here for retaining walls is analogous to the peak seismic coefficient used for landslide studies \( (k_{max}) \). See Section 4.5.2.4 for additional discussion on this issue for the landslide problem.
unrestrained backfill (i.e., \( f < V_s/8H \)), which encompasses the range of frequencies containing most of the seismic energy for many practical problems.

For tall walls where \( f > V_s/8H \), Steedman and Zeng’s results in Figure 131 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 130.

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. Nonplanar top surfaces and any surcharge loading need to be included in these analysis. 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/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 Equations 4.24-4.27.

4. Calculate the FSs for retaining wall stability.

5. 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, a number of issues should be considered when interpreting 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 reinforced soil walls, additional seismic analysis needs to be performed to include checks on internal stability such as a reinforcement pullout, reinforcement rupture and/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 Section 4.6).
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:

\[
\Delta P_{eq} = \gamma H^2 \frac{a_h}{g} F_P
\]

\[
\Delta M_{eq} = \gamma H^3 \frac{a_h}{g} F_M
\]

where

\( \Delta P_{eq}, \Delta M_{eq} = \) 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

The dimensionless dynamic thrust and moment factors are shown in Figure 128 and Figure 129, respectively. Wood recommends that the dynamic thrust be placed at a height of

\[
h_{eq} = \frac{\Delta M_{eq}}{\Delta P_{eq}}
\]

above the bottom of the wall (note that \( h_d = H - h_{eq} \)). This corresponds to a value of \( h_d = 0.37H \) for many cases.

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 Section 4.5.2.4.

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

\[
\bar{d}_{\text{perm}} = 37 \frac{v_{\text{max}}^2}{PHA} \exp \left( \frac{-9.4a_y}{PHA} \right)
\]  

(4.31)

where \(PHA\) is the peak horizontal ground acceleration (as used above), \(v_{\text{max}}\) 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:

\[
a_y = \left[ \tan \phi_{\text{backfill}} - \frac{P_{AE} \cos(\delta + \theta) - P_{AE} \sin(\delta + \theta)}{W} \right] g
\]

(4.32)

where \(W\) is the weight of the wall.

As with the Newmark analysis, the issues discussed in Section 4.5.2.4 are applicable to interpreting the results of this analysis.

Reinforced soil 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 reinforced soil walls and slopes are not routinely used. Nova-Roessig (1999) has proposed a deformation-based analysis for reinforced soil 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 Section 4.6 can be used to estimate the amount of seismic compression since no methods are currently available specific to reinforced soil walls.

Typical static design of retaining walls assume 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 (e.g., Figure 126). 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 likely to have not 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 do 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 seismically-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 (i.e., 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.

4.7.2 Damage, Repair, and Mitigation

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?

In this section, a brief overview of repair and mitigation strategies is provided. The conditions under which soil “damage” can occur are also provided. In this context, soil damage is defined as an earthquake-induced disruption of the residual strength of the soil.

4.7.2.1 Soil Damage from Retaining Wall Failure

Seismically-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 Section 4.7.1.4, interpretation of calculated pseudo static factors of safety and calculated seismic wall displacements must occur in the context of the condition of the wall, possible (small) design displacements that may have already occurred (e.g., Figure 126), and the collateral soil stresses induced by the static and seismic wall movement. When the seismically-induced wall movements are small, and analyses of future wall stability (described in Section 4.7.2.1) 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, it is possible that the backfill materials 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 is only possible in relatively clayey soil or clayey rock materials, and occurs because of clay fabric re-orientation such that clay particles are aligned with the direction of slip. This phenomenon would not be expected in non-plastic sands, silts, gravels, or bedrock comprised 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.

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.

4.7.2.2 Repair and Mitigation Strategies

Prior to making repair or mitigation recommendations, an engineer should have a clear understanding of site conditions, including the mode of failure 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. In addition, the engineer must be familiar with their recommended repair/mitigation technique and the technique’s limitations, risks, costs, and appropriateness for the site. Lastly, the engineer must understand their client’s needs and expectations.

The repair/mitigation measures recommended by an engineer must satisfy two criteria. First, the recommendations must be appropriate and consistent with the nature and extent of the retaining wall failure. Second, the recommendations must be consistent with an analysis of the possible future movement of the wall and the consequences of that movement. Appropriate repairs may address only damage to improvements or include mitigation measures as well, depending upon circumstances.

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 may 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 must be performed: (1) analysis of the wall system stability, (2) evaluation of structural condition, and (3) integrity of the drainage system.

Analysis of Wall System Stability

Wall system stability analyses should 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 follow typical design methodologies (ASCE, 1994). The results should be compared to minimum design standards (Table 4.5 on page 66). If the results of the analyses indicate that the wall is not adequately stable or serviceable, the engineer 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 Structural Condition

The 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 (see Chapter 5), proper mixing of components, and proper execution of the injection, crack repairs made from one-side only of the concrete element are effective in creating a repair that was comparable to the uncracked strength of concrete specimens that were free of restrained shrinkage stress (NAHB, 2002).

Integrity of Drainage System

Seismically-induced wall movements may damage the drainage system behind retaining walls. If this were to occur, 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 form 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 from the backfill soils. Evaluation of the drainage system integrity involves direct observation of drainage pipes and their connection 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.

Reinforced Soil Walls and Slopes

For reinforced soil 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.
Table 4.5 Summary of the typical factors of safety utilized for retaining wall design after ASCE (1994)

<table>
<thead>
<tr>
<th>Loading Condition</th>
<th>Usual</th>
<th>Unusual</th>
<th>Earthquake</th>
</tr>
</thead>
<tbody>
<tr>
<td>Base Sliding</td>
<td>1.5</td>
<td>1.33</td>
<td>1.1</td>
</tr>
<tr>
<td>Bearing Capacity</td>
<td>3</td>
<td>2</td>
<td>&gt;1</td>
</tr>
<tr>
<td>Overturning Criteria: minimum base area in compression</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Soil foundation</td>
<td>100%(^1)</td>
<td>75%(^1)</td>
<td>Resultant within base</td>
</tr>
<tr>
<td>Overturning Criteria: minimum base area in compression</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rock foundation</td>
<td>75%(^1)</td>
<td>50%(^1)</td>
<td>Resultant within base</td>
</tr>
</tbody>
</table>

\(^1\) 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.
4.8 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 may 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 life. 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/contraction of finishes. These movements are commonly manifested in the superstructure as finish cracking, foundation cracking, and uneven floors.

During post-earthquake reconnaissance of a flat site, an investigator will need to distinguish ground settlements and/or heave that are characteristic of static volume change phenomena from ground settlements associated with liquefaction or seismic compression. Similarly, investigations of sloping sites must distinguish long-term slope instability (landslides), creep, or retaining wall movements from ground deformations associated with seismically-induced landslides or retaining walls failure. The most common non-seismic modes of ground deformation or failure were listed in Section 4.1 as consolidation settlement, immediate settlement, hydro-compression settlement, expansive soil movement, landsliding, slope creep, and retaining walls failure. For each of these modes of deformation except landsliding and retaining wall failure, the following sections provide description of the phenomena, visual indications that distinguish the deformation from seismically-induced deformation, and briefly discuss investigative methods. Discussions of landsliding and retaining walls failure under static conditions are provided in Sections 4.5.2.3 and 4.7.1.3, respectively.

4.8.1 Consolidation Settlement

Consolidation settlement is defined as volume change due to dissipation of excess 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 (1994). The rate of consolidation settlement is dependent upon soil properties and length of the drainage path. 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 may be significant and take considerably longer to complete. The excess pore pressures causing consolidation may result from changes in overburden pressure (i.e., fill placement, addition of structural loads) or changes in ground water levels.

The rate of settlement is dependent upon soil properties and the length of the drainage path, and the amount of settlement depends on site conditions. 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

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Excess pore pressure is defined as pore pressures beyond the hydrostatic pore pressure.
floor elevation gradients, and doors and windows that may 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 and/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 may 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), and ASCE (1994)].

### 4.8.2 Immediate Settlement

Immediate settlement is defined as settlement caused by small-strain shear and/or volumetric deformations in soil that are not associated with consolidation or hydro-compression (discussed later). 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, but rather, depends principally on the bulk and shear modulus 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 may be found in ASCE (1994). Immediate settlements in cohesive soil may be estimated using elastic theory and are discussed in most geotechnical engineering texts [Terzaghi et al. (1996), Fang (1991), Lambe and Whitman (1969) and Bowles (1996)].

### 4.8.3 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 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 and/or the weakening of the chemical bonds (particularly acidic water). 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 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 often 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 possibly 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 and 105 lbs/ft$^3$, ASCE (1994). Investigations of hydro-compression settlement typically include laboratory testing of minimally disturbed soil samples or remoulded, 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 possible, 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 may be performed by several methods such as oedometer tests (ASTM D 4546) or via a modified oedometer test as described in ASCE (1994).

4.8.4 Expansive soil movement

Expansive soil movement is defined as shrink/swell of plastic clays when the water content is reduced (drying) or increased (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/shrinkage may occur because of changes in environmental conditions is termed the active zone.

Good indications that expansive soils are present at a site are 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, irrigation patterns, etc. 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. Typical damage and the effect of vegetation are schematically summarized in Figure 132 to Figure 135 for typical residential foundation systems.

Discussions of expansive soil behavior and the effect of vegetation may be found in ASCE (1994), Institute of Civil Engineers (1984), and BRE (1985). Identification of expansive soils may be performed with standard laboratory testing such as liquid limit and plastic limit (ASTM D 4318),

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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 (1994).

### 4.8.5 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 (freezing and thawing) and water content variations in the soil and rock within the surface layer and takes place primarily 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).

Slope creep occurs within shallow soil/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 (e.g., Figure 83). Figure 136 shows a schematic representation of damage that may result from creep.
American Society of Civil Engineers (ASCE) (1994). *Technical Engineering and Design Guides as Adapted from the US Army Corps of Engineers, No. 4, Retaining and Flood Walls*, ASCE Press, New York, NY.


Figures
Section 4.2 Figures
Figure 1  Surface fault rupture from the M 6.6 1971 San Fernando Earthquake and associated damage to pavement and structures in the Sylmar area. Courtesy of University of California, Berkeley, National Information Service for Earthquake Engineering.

Figure 2  Undamaged wood-frame building in fault graben from the M 7.0 1954 Dixie Valley-Fairview Peak, Nevada Earthquake. According to observations at the site, there was no broken glass, and the metal chimney was undamaged. The structure was on skids rather than permanent foundations. Courtesy of University of California, Berkeley, National Information Service for Earthquake Engineering.
Figure 3  Fence offset from the M 7.9 1906 San Francisco Earthquake shows 8.5 foot right lateral movement on the San Andreas fault near Woodville in Marin County north of San Francisco.

Figure 4  Surface faulting on Interstate 80 midway between El Centro and Holtville from the M 7.2 1940 El Centro, California earthquake. Courtesy of University of California, Berkeley, National Information Service for Earthquake Engineering.
Figure 5  Aerial view of faulting across a soccer field in Guatemala from the M 7.5 1976 Guatemala (Gualan) Earthquake. Courtesy of University of California, Berkeley, National Information Service for Earthquake Engineering.

Figure 6  Sidewalk displaced by distributed faulting in Managua, Nicaragua from the M 6.2 1972 Managua, Nicaragua earthquake. Courtesy of University of California, Berkeley, National Information Service for Earthquake Engineering.
Figure 7  Water line break along fault in Managua, Nicaragua from the M 6.2 1972 Managua, Nicaragua earthquake. Courtesy of University of California, Berkeley, National Information Service for Earthquake Engineering.

Figure 8  House damaged by surface fault rupture from the M 6.6 1971 San Fernando Earthquake. Courtesy of Applied Technology Council, ATC (1994).
Figure 9 Schematic illustrations of liquefaction induced instabilities. Source: Seed et al. (2001).
Figure 10  Schematic illustration of liquefaction induced lateral displacements.  Source: Seed et al. (2001).
Figure 11  Schematic illustration of damage to a building from lateral spreading. Courtesy of Applied Technology Council, ATC (1994).

Figure 12  Damage to a slab-on-grade from liquefaction at L.A. County Juvenile Hall, Sylmar, California during the M 6.5 1971 San Fernando Earthquake. Source: Martin and Lew (1999).
Figure 13  Settlement of residence in Simi Valley damaged as a result of soil liquefaction during the M 6.7 1994 Northridge Earthquake. Note settlement of door. Source: NAHB Research Center (1994).
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Strike-Slip Fault

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<table>
<thead>
<tr>
<th>RC</th>
<th>a</th>
<th>b</th>
<th>γ_v (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>90-92</td>
<td>1.2</td>
<td>0.8</td>
<td>0.04</td>
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<tr>
<td>93-95</td>
<td>0.8</td>
<td>0.8</td>
<td>0.04</td>
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Table 25.1 - Magnitude of Wall Rotation to Reach Failure

<table>
<thead>
<tr>
<th>SOIL TYPE AND CONDITION</th>
<th>ROTATION, Y/H*</th>
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<tr>
<td></td>
<td>ACTIVE</td>
<td>PASSIVE</td>
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<tr>
<td>Dense cohesionless</td>
<td>0.001</td>
<td>0.02</td>
</tr>
<tr>
<td>Loose cohesionless</td>
<td>0.004</td>
<td>0.06</td>
</tr>
<tr>
<td>Stiff cohesive</td>
<td>0.010</td>
<td>0.02</td>
</tr>
<tr>
<td>Soft cohesive</td>
<td>0.020</td>
<td>0.04</td>
</tr>
</tbody>
</table>

*Y = horizontal displacement and H = height of wall

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Schematic of Typical Interior Distress to Residential Units From Expansive Soils

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