Chapter 2
Soil Liquefaction in Earthquakes

2.1. Definition of Soil Liquefaction.

Soil liquefaction and related ground failures are commonly associated with large earthquakes. In common usage, *liquefaction* refers to the loss of strength in saturated, cohesionless soils due to the build-up of pore water pressures during dynamic loading. A more precise definition of soil liquefaction is given by Sladen et al. (1985):

> *"Liquefaction is a phenomenon wherein a mass of soil loses a large percentage of its shear resistance, when subjected to monotonic, cyclic, or shock loading, and flows in a manner resembling a liquid until the shear stresses acting on the mass are as low as the reduced shear resistance."*

In a more general manner, soil liquefaction has been defined as the transformation *"from a solid state to a liquefied state as a consequence of increased pore pressure and reduced effective stress"* ("Definition of terms..." 1978). Some ground failures attributed to soil liquefaction are more correctly ascribed to "cyclic mobility" which results in limited soil deformations without liquid-like flow. The proper, concise definition for soil liquefaction has been the subject of a continuing debate within the geotechnical profession. While investigators have argued that liquefaction and cyclic mobility should be carefully distinguished (Castro and Poulos 1977), "liquefaction" is commonly used to describe all failure mechanisms resulting from the build-up of pore pressures during undrained cyclic shear of saturated soils.

Liquefaction results from the tendency of soils to decrease in volume when subjected to shearing stresses. When loose, saturated soils are sheared, the soil grains tend to rearrange into a more dense packing, with less space in the voids, as water in the pore spaces is forced out. If drainage of pore water is impeded, pore water pressures increase progressively with the shear load. This leads to the transfer of stress from the soil skeleton to the pore water precipitating a decrease in effective stress and shear resistance of the soil. If the shear resistance of the soil becomes less than the static, driving shear stress, the soil can undergo large deformations and is said to liquefy (Martin et al. 1975; Seed and Idriss 1982). By the narrowest definition, true *liquefaction* refers only to the flow of soil under a static shear stress that exceeds the undrained, residual shear resistance of a contractive soil (Castro 1987). Liquefaction of loose, cohesionless soils can be observed under both monotonic and cyclic shear loads.

When dense sands are monotonically sheared, the soil skeleton may first compress and then dilate as the sand particles move up and over one another. For dense, saturated sands sheared without pore water drainage, the tendency for dilation or volume increase results in a decrease in pore water pressure and an increase in the effective stress and shear strength. When a dense sand sample is subjected to cycles of small shear strains under undrained conditions, excess pore pressure may be generated in each load cycle leading to softening and the accumulation of deformations. However, at larger shear strains, dilation relieves the excess pore pressure resulting in an increased shear resistance. The behavior of loose and dense sands in undrained shear is discussed further in Section 2.2.

Some investigators use the term "limited liquefaction" for conditions where large deformations after initial liquefaction are prevented by an increase in the undrained shear strength (Finn 1990). The propensity of dense, saturated sands to progressively soften in undrained cyclic shear, but achieve limiting strains under subsequent static loading, is more precisely described as *cyclic mobility* (Castro 1975: Castro and Poulos 1977). Cyclic mobility is distinguished from liquefaction by the fact that a liquefied soil exhibits no appreciable increase in shear resistance regardless of the magnitude of deformation (Seed 1979). Soils subject to cyclic mobility will first soften under cyclic loading but then stiffen when monotonically loaded without drainage as the tendency to dilate reduces the pore pressures. During cyclic mobility, the residual shear resistance remains greater than the driving static shear stress and deformations accumulate only during cyclic loading. However, in conventional usage, a soil failure actually resulting from cyclic mobility is often referred to as "liquefaction".

In addition, as pointed out by Selig and Chang (1981) and Robertson (1994), it is possible for a dilative soil to reach a temporary condition of zero effective stress and shear resistance. When the initial static shear stress is low, cyclic loads may produce a reversal in the shear stress direction. That is, the stress path passes through a state of zero shear stress. Under these conditions, a dilative soil may accumulate sufficient pore pressures to reach a condition of zero effective stress and large deformations may develop. However, deformations stabilize when cyclic loading ends because the tendency to dilate with further shearing increases the effective stress and shear resistance. Robertson (1994) termed this behavior *cyclic liquefaction*. Unlike cyclic mobility, cyclic liquefaction involves at least some deformation occurring while static shear stresses exceed the shear resistance (when the condition of zero effective stress is approached). However, deformations do not continue after cyclic loading ends as the tendency to dilate quickly results in strain hardening. Again, this type of failure in saturated, dense cohesionless deposits is usually identified as "liquefaction" but with limited deformations.

Considering these mechanisms of ground failure, Robertson (1994) and Robertson et al. (1994) suggested a fairly complete classification system to define "soil liquefaction". The latest version of this system for describing liquefaction, given by Robertson and Fear (1996), can be summarized as:

(1) *Flow liquefaction*, used for the undrained flow of a saturated, contractive soil when the static shear stress exceeds the residual strength of the soil. Failure may be triggered by cyclic or monotonic shear loading.

(2) *Cyclic softening*, used to describe large deformations occurring during cyclic shear due to pore pressure build-up in soils that would tend to dilate in undrained, monotonic shear. Cyclic softening, in which deformations do not continue after cyclic loading ceases, can be further classified as:

• *Cyclic liquefaction*, which occurs when cyclic shear stresses exceed the initial, static shear stress to produce a stress reversal. A condition of zero effective stress may be achieved during which large deformations may occur.

• *Cyclic mobility*, in which cyclic loads do not yield a shear stress reversal and a condition of zero effective stress does not develop. Deformations accumulate in each cycle of shear stress.

This classification system for liquefaction recognizes that various mechanisms may be involved in a given ground failure. Yet, this definition preserves the contemporary usage of the term "liquefaction" to broadly describe the failure of saturated, cohesionless soils during earthquakes.

Significant effort can be spent in devising nomenclature to clearly define the failure response of saturated soils in earthquakes. However, no definition or classification system appears to be entirely satisfactory for all possible failure mechanisms. While recognizing the complex soil behavior involved, a broad definition of soil liquefaction will be adopted for this study. As defined by the National Research Council's Committee on Earthquake Engineering (1985), *soil liquefaction* will be taken to include *"all phenomena giving rise to a loss of shearing resistance or the development of excessive strains as a result of transient or repeated disturbance of saturated cohesionless soils."*

2.2. Behavior of Saturated, Cohesionless Soils in Undrained Shear.

During an earthquake, the upward propagation of shear waves through the ground generates shear stresses and strains that are cyclic in nature (Seed and Idriss 1982). If a cohesionless soil is saturated, excess pore pressures may accumulate during seismic shearing and lead to liquefaction. In the laboratory, undrained, cyclic shear tests provide valuable insight into the behavior of saturated, cohesionless soils during earthquakes. Considerable research has been published on the behavior of liquefiable soils in such tests and excellent discussions are given by Castro (1975), Selig and Chang (1981), National Research Council (*Liquefaction...* 1985), Poulos et al. (1985), Sladen et al. (1985), Alarcon-Guzman et al. (1988), Ishihara (1993), Robertson (1994), and others. This section briefly reviews some of the more basic concepts helpful in understanding the liquefaction of soils.

Undrained monotonic and cyclic shear.

The behavior of a saturated soil under both monotonic and cyclic shear is depicted in Figure 2.1. The response of the same soil in loose (contractive) and dense (dilative) states is indicated in parts (a) and (b), respectively, of this figure. The tests depicted begin with an initial, static driving shear stress that represents the anisotropic stress conditions in a slope.

A loose soil tends to compact when sheared and, without drainage, pore water pressures increase. As indicated in Figure 2.1a, a contractive soil sheared monotonically reaches a peak shear strength and then softens, eventually achieving a residual shear resistance. If the residual shear strength is less than the static driving shear, a liquefaction flow failure results. If the same soil is sheared cyclicly, also depicted in Figure 2.1a, excess pore pressures are generated with each cycle of load. Without drainage, pore pressures accumulate and the effective stress path moves toward failure. If the shear strength falls below the static driving stress, a flow failure results and deformations continue after cyclic loading stops. For a liquefaction flow failure to occur, a saturated soil with a tendency to contract must undergo undrained shear of sufficient magnitude, or sufficient number of load cycles, for the shear resistance to become less than the static driving load. Under these conditions, tremendous deformations may occur before equilibrium conditions are re-established at the reduced shear strength.

Shearing of dense, dilative soils will also produce some excess pore pressure at small strains. However, at larger strains, the pore pressures decrease and can become negative as the soil grains, moving up and over one another, tend to cause an increase in soil volume (dilation). Consequently, as shown in Figure 2.1b, monotonic shearing of a dilative soil results in an increasing effective stress and shear resistance. Figure 2.1b also shows the response of the same dilative soil to dynamic loading. In this case, pore pressures are generated in each shear cycle resulting in an accumulation of excess pore pressure and deformation. However, beyond some point the tendency to dilate and develop negative pore pressures limits further straining in additional load cycles. As indicated in the bottom of Figure 2.1b, the effective stress path moves to the left but never reaches the failure surface. If the soil is sheared after the cyclic load ceases, the soil will develop the full strength that would be observed in a monotonic shear test. While significant strains can occur during cyclic loading, the very large deformations associated with a flow failure do not develop in dense, dilative soils. As introduced in Section 2.1, this behavior has been termed *cyclic mobility*. Hence, cyclic shear of dilative soils does not result in flow failures because, with undrained conditions, the shear strength remains greater than the static driving shear stress.

Steady-state concept.

Observations of soil behavior at very large shear strains have lead to the concept of a *steady state* as defined by Castro and Poulos (1977), Poulos (1981), and Poulos et al. (1985). A soil is in the steady-state condition if deformations are occurring at constant volume or void ratio, constant effective stress, constant shear stress or resistance, and constant rate of shear strain (velocity). In this condition, the original structure of the soil has been reworked at very high strains into a statistically constant particle orientation or "flow structure". A soil can reach the steady-state condition only after experiencing sufficient remolding and, possibly, particle breakage such that further deformations are not affected by particle orientation. A soil exists in the steady state only as long as deformations are occurring; if deformations cease, the soil is no longer considered to be in the steady-state condition. Steady-state flow can be achieved through drained or undrained, and monotonic or cyclic loading.

For a particular soil, a plot of possible conditions during steady-state flow produces a single curve in three dimensional space of void ratio, effective stress, and shear stress. This curve is the steady-sate line and can be plotted in the two projections depicted in Figure 2.2. The state diagram, shown in the top of Figure 2.2, is useful for predicting the volumetric response of a soil given the initial conditions of effective stress and void ratio. Regardless of the initial conditions, monotonic shearing of a soil leads to the steady-state line at sufficiently high strains. For conditions above the steady-state line on the state diagram, the soil will tend to contract when sheared producing excess pore pressures when drainage is impeded. For conditions below the steady-state line, the soil will tend to dilate and, without drainage, produce decreasing pore pressures. Drained shear tests follow vertical paths on the state diagram (constant effective stress) while undrained tests follow horizontal paths (constant void ratio).

For example, six monotonic shear tests (*A*-*H*) from three initial states are depicted on the state diagram in Figure 2.2. Drained shear of samples at two initial densities, but at the same effective confining stress (tests *A* and *B*), leads to the same point on the steady-state diagram and the same ultimate shear strength (s_2). For a sample at the same void ratio, a drained test at higher confining pressure (test *C*) yields a higher shear strength (s_4). In this case, tests *A* and *C* depict the shear of contractive samples while test *B* represents a dilative sample. Undrained monotonic shear tests (*D*-*F*) also lead to the steady-state line but along paths of constant void ratio. As shown in Figure 2.2, two undrained tests at the same void ratio (*E* and *F*) achieve the same shear strength at the steady-sate (s_3), while a test at a higher void ratio (*D*) leads to a lower stead-state strength (s_1). When undrained, the shear of contractive samples (tests *D* and *F*) generates excess pore pressures and decreasing effective stress paths, as indicated by the paths to the left on the state diagram. On the other hand, undrained shear of a dilative sample (test *E*) results in decreasing pore pressures and a path to the right.

Undrained cyclic loading tests can also be depicted on a state diagram, as illustrated by tests *G* and *H* in Figure 2.2. As discussed earlier, cyclic shear tends to produce excess pore pressures in both contractive and dilative soils resulting in paths of decreasing effective stress on the state diagram. Hence, cyclic mobility of a dilative soil (test *H*) produces a path away from the steady-state line. Subsequent monotonic shearing of dilative or contractive samples follows a path toward the steady-state line.

Hence, the steady-state line is useful for distinguishing between combinations of density (represented by void ratio) and effective stress producing contractive or dilative states, as well as the relative magnitude of the shear resistance in steady-sate flow. Conditions placing a soil above its steady-state line (high consolidation pressure or high void ratio) indicate that the soil will tend to contract and is subject to possible liquefaction under dynamic shear loads. Under either monotonic or cyclic loads, a contractive soil exhibits the same shear resistance in the steady-state condition. Low consolidation pressures or dense packing of the soil grains places the soil below the steady-state line where the soil might be subject to cyclic mobility but is not prone to a liquefaction flow failure. That is, cyclic loading of a dilative soil may induce limited shear deformations without approaching a steady-state flow condition.

2.3. Susceptibility of Soils to Liquefaction in Earthquakes.

Liquefaction is most commonly observed in shallow, loose, saturated deposits of cohesionless soils subjected to strong ground motions in large-magnitude earthquakes. Unsaturated soils are not subject to liquefaction because volume compression does not generate excess pore pressures. Liquefaction and large deformations are more likely with contractive soils while cyclic softening and limited deformations are associated with dilative soils. As discussed in the previous section, the steady-state concept demonstrates how the initial density and effective confining stress affect the liquefaction characteristics of a particular soil. Other factors affecting the liquefaction susceptibility of different soil types are discussed in this section. In practice, the potential for liquefaction in a given soil deposit during an earthquake is often assessed using in situ penetration tests and empirical procedures. The most widely accepted procedure for evaluating liquefaction susceptibility, based on the Standard Penetration Test, is reviewed in Chapter 7.

Since liquefaction is associated with the tendency for soil grains to rearrange when sheared, anything that impedes the movement of soil grains will increase the liquefaction resistance of a soil deposit. Particle cementation, soil fabric, and aging - all related to the geologic formation of a deposit - are important factors that can hinder particle rearrangement (Seed 1979). Soils deposited prior to the Holocene epoch (more than 10,000 years old) are usually not prone to liquefaction (Youd and Perkins 1978), perhaps due to weak cementation at the grain contacts. However, conventional sampling techniques inevitably disturb the structure of cohesionless soils such that laboratory test specimens are usually less resistant to liquefaction than the in situ soil. Even with reconstituted laboratory samples, the soil fabric and resistance to liquefaction are affected by the method of preparation such as dry pluviation, moist tamping, water sedimentation, etc. After liquefaction has occurred, the initial soil fabric and cementation have very little influence on the shear strength beyond about 20% strain (Ishihara 1993).

Stress history also plays an important role in determining the liquefaction resistance of a soil. For example, deposits with an initial static shear stress (anisotropic consolidation conditions) are usually more resistant to pore pressure generation (Seed 1979), although static shear stresses may cause greater deformations once liquefaction occurs. Stress history may also contribute to the liquefaction resistance of older deposits. Overconsolidated soils, having been subjected to greater static pressures in the past, are more resistant to particle rearrangement and liquefaction. Soil deposits subjected to past cyclic loading are usually more resistant to liquefaction as the soil grains tend to be in a more stable arrangement, but some deposits may be loosened by previous shaking.

In addition, the frictional resistance between soil grains is proportional to the effective confining stress. Consequently, the liquefaction resistance of a soil deposit increases with depth as the effective overburden pressure increases. For this reason, soil deposits deeper than about 15 m are rarely observed to liquefy (Krinitzsky et al. 1993).

Characteristics of the soil grains (distribution of sizes, shape, composition, etc.) influence the susceptibility of a soil to liquefy (Seed 1979). While liquefaction is usually associated with sands or silts, gravelly soils have also been known to liquefy. Rounded soil particles of uniform size are generally the most susceptible to liquefaction (Poulos et al. 1985). Well-graded sands with angular grain shapes are generally less prone to liquefy because of a more stable interlocking of the soil grains. On the other hand, natural silty sand sediments tend to be deposited in a looser state, and thus are more likely to exhibit contractive shear behavior, than clean sands.

Clays with measurable plasticity are resistant to the relative movement of particles during cyclic shear loading and are generally not prone to pore pressure generation and liquefaction. Plastic fines in sandy soils usually create sufficient adhesion between the sand grains to limit the ability of larger particles to move into a denser arrangement. Consequently, soils with a significant plastic fines content are rarely observed to liquefy in earthquakes. In contrast, as discussed by Ishihara (1993), non-plastic soil fines with a dry surface texture (such as rock flour) do not create adhesion and do not provide significant resistance to particle rearrangement and liquefaction. Moreover, low plasticity fines may contribute to the liquefaction susceptibility of a soil. Koester (1994) suggests that sandy soils with a significant fines content may be inherently collapsible, perhaps due to the greater compressibility of the fines between the sand grains.

Permeability also affects the liquefaction characteristics of a soil deposit. When pore water movement within a liquefiable deposit is retarded by a low permeability, pore pressures are more likely to accumulate during cyclic loading. Consequently, soils with a large non-plastic fines content may be more susceptible to liquefaction because the fines inhibit drainage of excess pore pressures. In addition, the liquefaction vulnerability of a soil deposit is affected by the permeability of surrounding soils. Less pervious clayey soils can prevent the rapid dissipation of excess pore pressures generated in an adjacent deposit of saturated sand. On the other hand, sufficient drainage above or below a saturated deposit may prevent the accumulation of pore pressures and liquefaction. Due to a relatively high permeability, gravelly soils are less prone to liquefy unless pore water drainage is impeded by less pervious, adjoining deposits.

2.4. Ground Failure Resulting from Soil Liquefaction.

Once the likelihood of soil liquefaction has been identified, an engineering evaluation must focus on the mode and magnitude of ground failures that might result. The National Research Council *(Liquefaction...* 1985) lists eight types of failure commonly associated with soil liquefaction in earthquakes:

- *Sand boils*, which usually result in subsidence and relatively minor damage.
- *Flow failures of slopes* involving very large down-slope movements of a soil mass.
- *Lateral spreads* resulting from the lateral displacements of gently sloping ground.
- *Ground oscillation* where liquefaction of a soil deposit beneath a level site leads to back and forth movements of intact blocks of surface soil.
- *Loss of bearing capacity* causing foundation failures.
- *Buoyant rise of buried structures* such as tanks.
- *Ground settlement*, often associated with some other failure mechanism.
- *Failure of retaining walls* due to increased lateral loads from liquefied backfill soil or loss of support from liquefied foundation soils.

The nature and severity of liquefaction damage is a function of the reduced shear strength and the magnitude of the static shear loads supported by the soil deposit (Ishihara et al. 1991). Castro (1987) classifies the possible consequences of liquefaction, as shown in Table 2.1, based on the relative magnitude of static driving shear stresses that may be present due to a surface slope or a foundation bearing load. When the driving shear loads are greater than the reduced strength of a liquefied soil deposit, a loss of stability can result in extensive ground failures or flow slides. However, if the driving shear stresses are less than the shear strength (perhaps due to dilation at large strains) only limited shear deformations are likely. On level ground with no driving shear stresses, excess pore pressures may break through to the surface to form sand boils; while the venting of liquefied soil may cause settlements, damages are usually not extensive in the absence of static shear loads.

Ground failures associated with liquefaction under cyclic loading can be broadly classified as (*Liquefaction...* 1985; Robertson et al. 1992):

(1) *Flow failures*, occurring when the liquefaction of loose, contractive soils (that do not gain strength at large shear strains) results in very large deformations.

(2) *Deformation failures*, occurring when a liquefied soil gains shear resistance at large strains, yielding limited deformations without loss of stability.

Whitman (1985) suggests the terminology *disintegrative* versus *non-disintegrative* failures, respectively, for these two categories. Considerable effort can be spent in devising specific terminology to categorize ground failures related to cyclic loading of saturated soils. Many investigators restrict the application of the term "liquefaction" to describe the loss of static resistance in truly undrained shear. While limited ground movements in earthquakes may be more accurately attributed to cyclic mobility, the term "liquefaction" is frequently used in a broader sense to describe this phenomena (Seed and Idriss 1971). In an effort to limit confusion in terminology, Whitman (1985) proposed the more general term *liquifailure* to cover all failures associated with cyclic loading of saturated, cohesionless soils.

However, it is common practice to describe all types of ground failure resulting from pore pressure generation and loss of shear strength in soils during cyclic loading as *liquefaction*. Thus, *liquefaction-induced* lateral spreading will be used in this report to describe ground failures that involve limited deformations. This is consistent with the definition of soil liquefaction adopted in Section 2.1.

Table 2.1. Classification of soil liquefaction consequences (after Castro 1987).

In Situ Stress Condition	Soil Behavior	Typical Field Observation
No driving shear stress	• Volume decrease • Pore pressure increase	• Ground settlement • Sand boils and ejection from surface fissures
Driving shear greater than residual strength	• Loss of stability • Liquefaction	• Flow slides • Sinking of heave buildings • Floating of light structures
Driving shear less than residual strength	• Limited shear distortion • Soil mass remains stable	• Slumping of slopes • Settlement of buildings • Lateral spreading

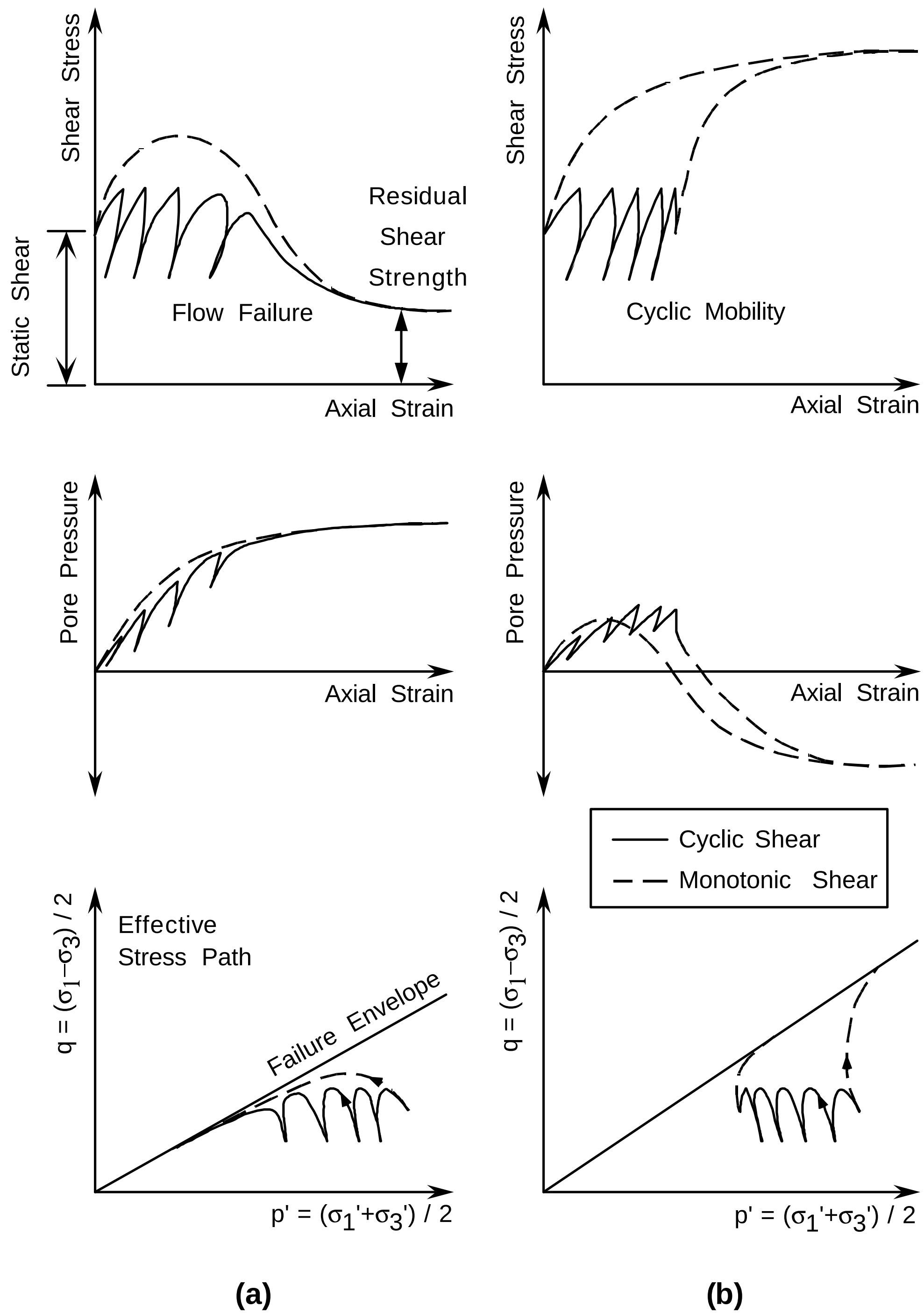

Figure 2.1. Response of (a) contractive and (b) dilative saturated sands to undrained shear.

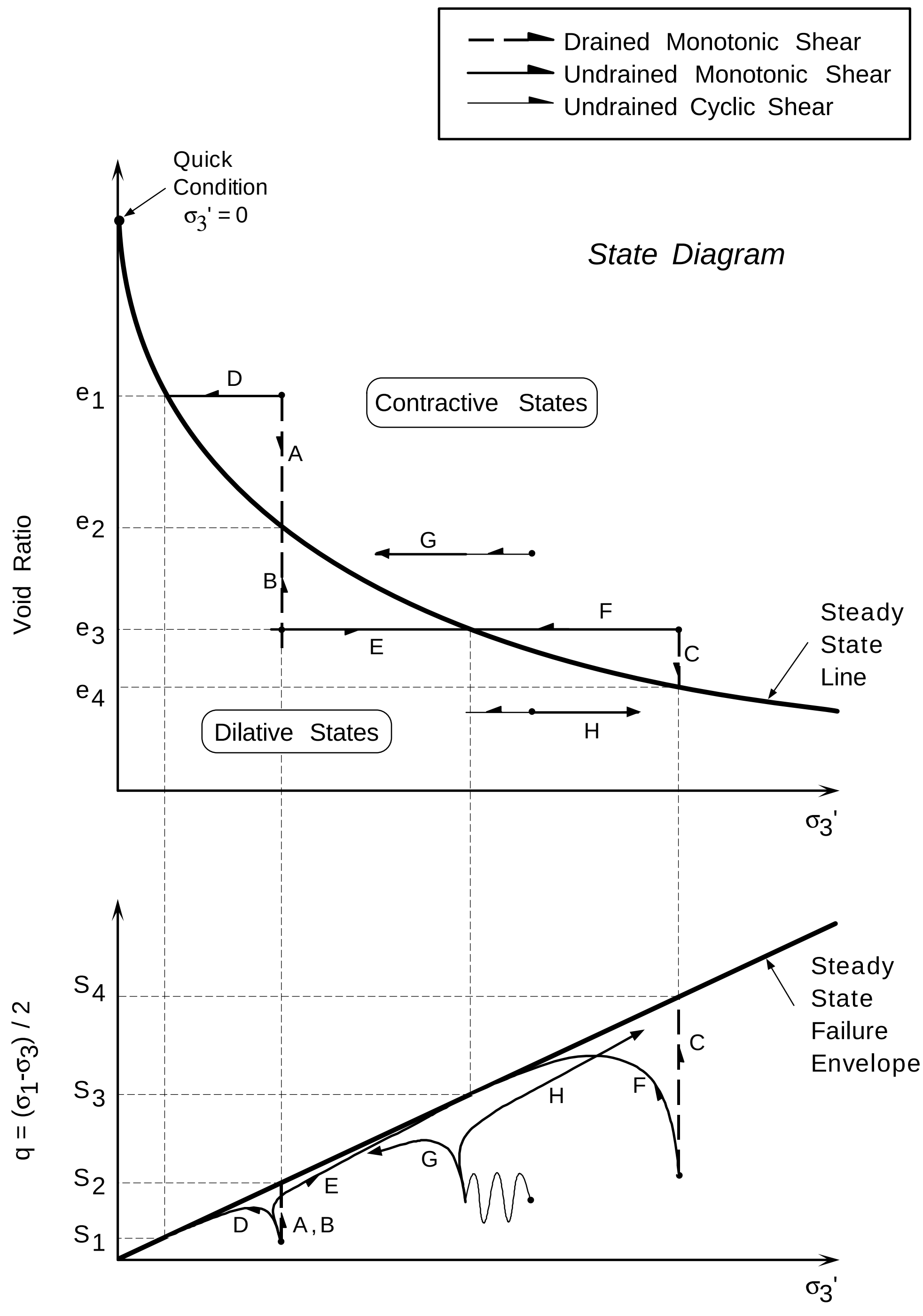

Figure 2.2. Monotonic and cyclic shear paths on a state diagram.