Chapter 3

Liquefaction-Induced Lateral Spreading

3.1. Description of Lateral Spreading.

Liquefaction-induced lateral spreading is defined as the finite, lateral displacement of gently sloping ground as a result of pore pressure build-up or liquefaction in a shallow underlying deposit during an earthquake. Liquefaction-induced lateral spreading is depicted schematically in Figures 1.1 and 3.1.

In a classification system for ground deformations and slope failures, Varnes (1978) describes lateral spreading with:

> *"Movements may involve fracturing and extension of coherent material . . . owing to liquefaction or plastic flow of subjacent material. The coherent upper units may subside, translate, rotate, or disintegrate, or they may liquefy and flow. The mechanism of failure can involve elements not only of rotation and translation but also of flow. . ."*

Restricting the consideration to ground deformations resulting from soil liquefaction in earthquakes, liquefaction-induced lateral spreading has been defined as the *"lateral displacement of large, surficial blocks of soil as a result of liquefaction in a subsurface layer"* (*Liquefaction...* 1985). Note that liquefaction beneath moderate to steep slopes can result in extensive flow slides involving tremendous down-slope movements or "flow" of completely broken soil over relatively long distances. Here, lateral spreading refers to the more moderate movements of gently sloping ground due to soil liquefaction. As discussed in Sections 3.4 through 3.8, the magnitude of lateral spreading deformation is affected by a complex interaction of many factors.

As described by Bartlett and Youd (1992a; 1992b), liquefaction-induced lateral spreading occurs on mild slopes of 0.3 to 5% underlain by loose sands and a shallow water table. Such soil deposits are prone to pore pressure generation, softening, and liquefaction during large earthquakes. If liquefaction occurs, the unsaturated overburden soil can slide as intact blocks over the lower, liquefied deposit. As depicted in Figure 3.1, surface displacements proceed down-slope or toward a steep free face (such as a stream bank) with the formation of fissures, scarps, and grabens. The geologic conditions conducive to lateral spreading (gentle surface slope, shallow water table, liquefiable cohesionless soils) are frequently found along streams and other waterfronts in recent alluvial or deltaic deposits, as well as in loosely-placed, saturated, sandy fills (Youd and Hoose 1976).

Horizontal displacements in a lateral spread can range up to several meters with smaller associated settlements. Characteristic patterns of ground deformation are evident from numerous lateral spreads described in the literature (see Chapter 6). At the head of a slide, ground fissures or tension cracks are usually found running perpendicular to the direction of slope movement. Ground fissures are also frequently found around the upper margins of the slide area and may be curved. Subsidence typically occurs at the head of a lateral spread with heaving at the toe. Sand boils, a common indication of soil liquefaction, are more frequently observed in the lower portions of a lateral spread.

Lateral spreading is defined here to include only lateral sliding of gently sloping ground due to soil liquefaction at relatively shallow depths, and does not refer to the large horizontal flows associated with deep-seated liquefaction failures. This definition of lateral spreading also specifically excludes two types of liquefaction-induced ground failure that can produce similar patterns of surface movements. As a result of liquefaction, slumping of embankments and tilting of retaining walls produce horizontal movements and ground fissures; accordingly, these failures are frequently described as "lateral spreading" in post-earthquake damage surveys. For example, the failure of highway and railroad embankments built over liquefiable soils is characterized by settlements, lateral displacements, and surface cracks or scarps running parallel to the slope face. Although liquefaction results in "lateral spreading" of the embankment, the slope failure is more accurately described as a rotational slide or slumping (Varnes 1978). In addition, the outward movement of retaining walls due to liquefaction in the backfill or foundation soils results in ground deformations behind the wall that are often described as "lateral spreading" (Ishihara et al. 1996). This type of earthquake failure is very common at harbor facilities where loose, saturated fills are found behind quay walls. However, these failures are more correctly associated with lateral earth pressures and performance of the wall. To reiterate, the definition of *lateral spreading* used in this report *does not* include failures of embankments and retaining walls due to soil liquefaction.

3.2. Impact of Lateral Spreading on Civil Infrastructure.

With horizontal displacements ranging up to several meters in lateral spreads, constructed facilities of most types are vulnerable to heavy damage. Structures at the head of the slide are sometimes pulled apart while those at the toe are subject to buckling or compression of the foundation soils. Differential settlements or heaving of the ground surface can also damage structures. While large ground movements can collapse buildings, horizontal and vertical displacements of lesser magnitude can lead to severe structural damage.

As discussed in Section 1.1, lateral spreading has caused considerable damage to civil infrastructure in past earthquakes. The high economic losses attributed to lateral spreading result mostly from the disruptive nature of individual failures at multiple sites over large geographic areas. In addition, lateral spreads frequently occur on fairly flat sites beside streams and other waterfronts where saturated, recent sediments are commonly found. Waterfront property on nearly level ground is also highly desirable for development. To some degree, the high damage costs associated with liquefaction-induced lateral spreading are related to the value of urban and industrial development located on sites prone to lateral spreading.

Linear infrastructure, such as utility lines and transportation routes, is particularly susceptible to damage in earthquakes from lateral spreads at multiple locations. Facilities of this type are often considered to be *lifelines*, which are expected to continue uninterrupted operation in the event of an earthquake. Because individual lateral spreads tend to occur over relatively large regions, utility and transportation routes can be impacted at numerous locations. O'Rourke and Lane (1989) point out that lateral spreading is often severely disruptive to lifeline facilities because:

(1) lateral spreads are rather common in developed areas during large earthquakes;
(2) lateral spreads occur on gentle slopes that appear stable and the potential hazard is not recognized;
(3) even when a site is identified as susceptible to lateral spreading, ground deformation patterns are difficult to predict; and,
(4) for buried pipelines, movements of surficial blocks of unliquefied soil induce large loads on the pipe.

Transportation routes are disrupted when lateral spreading causes damage to pavements, misalignment of railroad tracks, or the failure of bridge piers and abutments. However, buried pipelines are especially vulnerable to damage from liquefaction and lateral spreading:

> *"Pipelines usually are constrained to follow rights-of-way or otherwise conform to existing properties, so that poor ground conditions and potentially unstable sites cannot always be avoided. These constraints can be important for large transmission pipelines which cross broad geographical areas. Accordingly, the design of a pipeline in seismic zones may need to account for large soil displacements, especially if the pipeline is located near or within an area of potential soil liquefaction. Lateral spreads are among the most troublesome liquefaction hazards for buried pipelines"* (O'Rourke and Lane 1989).

Lateral spreads are often the primary earthquake hazard to regional pipeline networks and, in appraisals of seismic risk, frequently determine system performance (Ballantyne 1994; Honegger 1994). In evaluating the impact on a buried pipeline, the magnitude of displacements, direction and pattern of deformations, and areal extent of a potential lateral spread are critical design considerations.

Different orientations of a pipeline crossing a lateral spread, which determine the nature of loads imparted to the pipe, are illustrated in Figure 3.2. A buried pipe in a perpendicular crossing (Figure 3.2a) is subjected to bending in which the magnitude of ground displacement is a critical design factor. In a parallel crossing (Figure 3.2b), a pipeline is subjected to axial, frictional forces imposed by the soil moving relative to the pipe. Since the peak frictional force per unit length is mobilized with relatively small movements, the total axial load in a parallel crossing is principally determined by the length of pipe crossing the lateral spread. Many pipelines can tolerate large tensile and bending loads but are susceptible to buckling failures in compression; consequently, a parallel crossing is often the worst-case geometry (Honegger 1994; O'Rourke and Liu 1994). Moreover, the capacity of a pipeline to survive large deformations depends on the type of construction. Continuously welded steel transmission pipes are often designed to accommodate more than 5 meters of transverse displacement (Honegger 1992). On the other hand, jointed pipes of weaker materials may fail from displacements of only 10 cm.

Mitigation strategies.

If liquefaction and lateral spreading are likely in the event of an earthquake, anticipated ground displacements may indicate the need for mitigation efforts. Strategies to protect infrastructure from damage due to lateral spreading include:

(1) Relocate all facilities to locations outside the area of lateral spreading. This may not be an option for an existing structure or utility confined to a right-of-way.

(2) Ground improvements, such as densification, to prevent liquefaction and lateral spreading at a site. When a large area requires treatment, this approach is often uneconomical.

(3) Support lateral spreading deformations in relatively narrow zones around affected structures. Slope movements can be buttressed with piles, in-ground retaining walls, or ground improvements (such as in situ densification, grouting, deep soil mixing, stone columns, or other appropriate technology) in areas adjacent to the affected structure. In each case, the goal is to isolate the affected structure or buried utility from lateral movements of the surrounding ground. Studies by Yasuda et al. (1992b) and Towhata et al. (1996) have shown that continuous walls or zones of compacted soil built across a lateral spread can be effective in reducing ground displacements.

(4) Absorb ground deformations up-slope of vulnerable structures. Conceivably, movements could be absorbed by trenches, deformable walls, or other similar means.

(5) Construct or modify the structure to survive ground movements while remaining in service. Pipelines can be designed to move with the ground without breaking or foundations can be constructed to withstand anticipated soil displacements.

Options 4 and 5 require predictions of ground displacements, spurring the need for reliable means of modeling lateral spreading.

3.3. Scale Model Simulations of Lateral Spreading.

Laboratory-scale models, built on conventional shake tables and geotechnical centrifuges, have been used to study lateral spreading under controlled conditions. Model tests are useful for studying the onset of liquefaction and, qualitatively, some aspects of post-liquefaction behavior in lateral spreads. Unfortunately, state-of-the-art shake table and centrifuge tests do not appear to be capable of simulating all relevant aspects of lateral spreading. Hence, while useful in exploring the mechanics of liquefaction failures, laboratory scale models are unsatisfactory in providing fully realistic models of lateral spreading.

Shake table models.

Over the last several years, Japanese researchers have conducted a series of large shake table tests intended to model the behavior of liquefaction-induced lateral spreads. These tests have been reported by Towhata et al. (1989; 1991), Sasaki et al. (1991; 1992), Yasuda et al. (1991a; 1992a), and Tokida et al. (1993). The models tested were generally 1 m by 1 m or 0.8 m by 6 m in plan and up to 1.25 m in depth, although some models were smaller. The scale models were constructed on shake tables and liquefied with cyclic horizontal motions on the base. Variations in surface slope, bottom slope, thickness of soil layers, soil density, and other factors were studied. Qualitatively, the behavior of these models match field observations; namely, surface displacements are oriented down-slope, tension cracks and subsidence develop at the head of the slide, and, in the absence of a free face, smaller displacements and heaving occur near the toe of the slope.

A shake table model, however, can not simulate every detail of a liquefaction event in the field. Of particular importance, because the models are limited in depth, these tests do not realistically simulate the stress conditions present in natural slopes (Arulanandan and Scott 1993). Since the driving gravity stresses vary with the thickness of soil above the liquefied deposit, the static shear stresses in a scale model of a lateral spread will be lower than that which exists in the field. Moreover, a soil placed in a model at the same density as in the field, but at lower overburden stresses, will exhibit a lower ultimate shear resistance. Toyota and Towhata (1994) recognized that if a soil is contractive in the field, the same soil at the same density in a model under lower overburden pressure may be dilative. Therefore, the liquefied soil behavior in a shake table model may differ significantly from the field response. For these reasons, shake table models might not realistically simulate conditions where the static stresses exceed the residual strength of a liquefied soil deposit (Ishihara et al. 1991).

Also problematic, shorter drainage paths in a scale model result in more rapid pore pressure dissipation rates than found in the field. Consequently, the liquefied soil in a scale model tends to re-solidify more rapidly, resulting in smaller displacements, when the model is subjected to the same earthquake excitation as a lateral spread in the field. Short drainage paths in a scale model thus impair simulation of displacement magnitudes in a lateral spread. Because of this limitation, model tests are often performed and interpreted to give the maximum possible displacements that would occur if seismic shaking continued for a fairly long period of time. Thus, the resulting displacements often exceed the deformations likely to occur in a lateral spread during an earthquake. In the shake table tests reported by Sasaki et al. (1992), model shaking continued after the soil was liquefied until all slope movements stopped; in the field, this would represent a very long duration earthquake. In other model tests, the base excitations were continued for an arbitrary time of 10 to 40 sec after the soil liquefied (Miyajima et al. 1991; Sasaki et al. 1991; 1992; Yasuda et al. 1991a; 1992a). More recently, Hamada et al. (1994) conducted a test where a level model was liquefied, shaking was then stopped, and the model was tilted to induce gravity flow.

Capillary rise can also be a significant factor at the scale of a laboratory model. Capillary action creates negative pore pressures that, because of the increased effective stress, adds some finite shear strength to the soil. Since capillary rise can affect a significant thickness of soil in a model, it is difficult to simulate conditions of unsaturated soil over liquefiable soil, as commonly found in the field (Sasaki et al. 1992). To control capillary rise, a thin layer of coarse soil can be placed in the model, although this can introduce yet another factor that is not representative of field conditions.

Finally, the rigid walls of a shake table container may alter the behavior of the model along the edges (Sasaki et al. 1992). This problem can be mitigated by lining the inside of the model box with foam rubber inserts (Yasuda et al. 1992a) or constructing the model box with stacked rings that are free to slide horizontally over one another.

Centrifuge models.

Geotechnical centrifuge equipment allows for the correct simulation of stress conditions in scale models of soil structures. The model is constructed at a dimensional scale of 1/N and placed in a centrifugal acceleration field of N g's, produced by rotating the model at high speed. The resulting radial stresses in the soil model are then identical to the vertical, static stresses in a prototype or real-world soil structure (Kutter 1984; Fiegel and Kutter 1994a; 1994b). For simulations of liquefaction failures, a scale model can be constructed on a shake table mounted inside the centrifuge. After reaching an equilibrium condition in the rotating model, the base of the model is shaken horizontally to simulate an earthquake. If all of the pertinent scaling laws are satisfied, scale models in a centrifuge can more accurately simulate the behavior of full-scale earth structures.

Fiegel and Kutter (1994a; 1994b) and Elgamal et al. (1996) have performed several centrifuge model tests to simulate liquefaction-induced lateral spreading of gently sloping ground. These tests have been useful in illustrating the pattern of displacements across a vertical section through the liquefied soil as discussed in Section 3.6.

In the VELACS project (Verification of Liquefaction Analysis by Centrifuge Studies), the National Science Foundation has supported a coordinated study involving centrifuge simulations of liquefaction events at eight universities (Arulanandan and Scott 1993). To evaluate the repeatability and reliability of the centrifuge tests, each participating laboratory conducted a comparison-standard model test. Even with some scatter in the results, it was concluded that such tests could be used to verify numerical calculation procedures. However, centrifuge models of lateral spreading can suffer from the same deficiencies encountered in 1-g shake table models such as short drainage paths, capillary rise, and rigid container walls. Moreover, centrifuge models of lateral spreading are hampered by two additional, significant problems.

First, Arulanandan and Scott (1993) report difficulties in accurately determining displacements in the VELACS centrifuge tests. Very small displacements occurring in the centrifuge model correspond to large movements in a real slope; this necessitates very precise displacement measurements in the scale model. Because it is difficult to devise a reference point that moves with the soil after liquefaction without affecting the displacement pattern, precise displacement measurements are difficult to obtain. Moreover, under the influence of the radial acceleration field, a liquefied soil flows toward a curved surface matching the radius of the centrifuge. The initial model must be constructed precisely to match this curvature or the observed deformation of the model slope will not be representative. Because of these difficulties, Arulanandan and Scott consider measured displacements to be inherently less accurate than all other measured data in centrifuge models of liquefaction events.

A second drawback with centrifuge models arises from the difficulty in scaling the time rate of pore water seepage. If water is used to saturate the model, pore pressure dissipation is much more rapid over the smaller drainage paths in the model than would occur at full scale in the prototype. For a model accelerated to N g's, seepage and pore pressure dissipation will proceed as if the soil in the prototype had a permeability N times greater than the soil in the centrifuge model (Fiegel and Kutter 1994a; 1994b). Hence, a model constructed of silt might have the seepage characteristics of sand in the field. Simulating the liquefaction and lateral spreading of real soils is thus problematic. One way to address this issue is to saturate the centrifuge model with a fluid of greater viscosity, such as silicone oil or a glycerine-water mix. However, a pore fluid other than water may adversely affect the simulation in other ways. Several investigators have been unable to generate catastrophic flow failures in centrifuge models that were saturated with silicone oil (Arulanandan et al. 1988).

3.4. Behavior of Liquefied Soil in Lateral Spreads.

Contractive versus dilative soil response.

The magnitude of ground displacements in a liquefaction failure is influenced by the volumetric response of the liquefied soil. A contractive soil response can lead to very dramatic failures while a dilative response tends to limit the magnitude of displacements.

As discussed in Section 2.2, liquefaction leading to massive displacements in a flow slide involves the undrained shear of saturated, contractive soils that flow in a steady-state condition (Poulos et al. 1985). A flow failure can occur if the shear resistance of the soil in the steady-state condition is less than the static driving shear stress, as shown in Figure 3.3a. Hence, flow slides are usually associated with large, deep-seated slides where, in the presence of large static driving shear stresses, high confining pressures produce a contractive soil response (Ishihara et al. 1991).

Conversely, the more limited displacements in lateral spreads are associated with shallow soil deposits and gentle slopes which create lower confining pressures and static shear stresses. A low confining pressure can result in dilative behavior, even for a somewhat loose packing of the soil grains, as indicated by a state diagram (Figure 2.2). In the presence of static shear, a dilative soil can undergo cyclic mobility and cause limited slope movements during an earthquake (Castro 1975; Poulos et al. 1985). A soil exhibiting cyclic mobility may exhibit significant shear deformations but without reaching a steady-state condition. Under these conditions, a flow slide does not develop because the soil tends to dilate and gain strength after cyclic loading ends, as depicted in Figure 3.3b. On the other hand, low static shear stresses may preclude a flow failure in a contractive soil as also indicated in Figure 3.3b.

Undrained residual shear strength.

The shear resistance of the liquefied soil deposit in a lateral spread will clearly influence the magnitude of the ultimate displacements. However, it is not clear how to quantify the relationship between the changing shear resistance of the liquefied soil and the ultimate displacements. The residual shear strength during steady-state deformation of a soil, commonly used to evaluate post-liquefaction stability, is difficult to estimate and may not apply to lateral spreading failures.

As shown in Figures 2.1a and 3.3a, a contractive soil subject to liquefaction in undrained shear may experience a flow failure at a residual shear strength. In evaluating the post-earthquake stability and deformation of embankments, it is difficult but often critical to determine the residual shear strength of liquefiable soils (Ishihara et al. 1991; Mitchell 1993). Consequently, estimation of the residual shear strength of liquefied soils has been the subject of considerable research in recent years. As discussed below, estimating the undrained steady-state shear strength for in situ deposits is "*still a difficult task to achieve with a reasonable degree of credibility*" (Ishihara 1994).

The undrained residual shear strength (S_{us}) is the minimum shear resistance mobilized during steady-state deformations (Ishihara 1994). The residual strength of a liquefied soil can be estimated from monotonic, undrained shear tests following a procedure proposed by Poulos et al. (1985). This procedure utilizes a state diagram and attempts to correct for unavoidable changes in the density of field samples sheared in the laboratory. Considerable judgment is required to compute accurate and precise void ratios for the in situ deposit and high quality field samples, as demonstrated by Castro et al. (1992). Unfortunately, small changes in void ratio translate into large differences in the estimate of S_{us}, making this procedure sensitive to small errors. Consequently, it is difficult to get a reliable estimate of S_{us} from the state diagram (Seed and Harder 1990; Ishihara 1994). Furthermore, other studies (Marcuson et al. 1990; Stark and Mesri 1992; Vaid and Thomas 1995) indicate that the steady-state line is affected by sample preparation techniques, orientation of bedding planes, effective confining pressure, and mode of shear loading. These factors violate, to some degree, the basic assumptions in the procedure proposed by Poulos et al. (1985). Finally, the movement of pore water in a liquefied soil deposit alters the void ratio leading to large changes in the residual strength; hence, S_{us} values determined from undrained tests in the lab tend to give poor representations of field behavior (Seed 1987; Towhata 1991; Sasaki et al. 1992).

As an alternative to using laboratory tests, empirical correlations between S_{us} and the in situ penetration resistance have been suggested. Seed (1987) and Seed and Harder (1990) proposed a correlation with the SPT penetration resistance based on values of S_{us} back-calculated from several liquefaction failures. Similar data relating a normalized S_{us} to the cone penetrometer tip resistance have been published by Ishihara (1994). Unfortunately, these correlations show considerable data scatter. As discussed by Stark and Mesri (1992), partial drainage during failure increases the shear resistance of field deposits and back-calculated values of S_{us} may be more representative of the shear resistance during a flow failure than the lower residual shear strength when liquefaction was triggered. Consequently, correlations with in situ test data are generally based on ranges of S_{us} estimated for ground failures in the field.

Given the difficulty in estimating S_{us} for a liquefied soil deposit, it is difficult to quantify the influence of the residual shear strength on lateral spreading. More significantly, lateral spreading may not involve steady-state flow of liquefied soil. As indicated in Figure 3.3b, lateral spreading of gently sloping ground may be more commonly associated with limited deformations of dilative soils or contractive soils under low static shear loads. Therefore, the undrained residual shear strength of the liquefied deposit is may not be a strong indicator of displacement magnitudes in a lateral spread.

Influence of fines content.

As noted earlier in Section 2.3, plastic or non-plastic fine particles in a coarse-grained soil matrix affect the liquefaction characteristics of a soil. As discussed below, both the liquefaction resistance and the residual shear strength of a soil deposit are influenced by the fines content (percent by weight finer than 0.075 mm). In addition, finer particles impede the drainage of pore water and dissipation of excess pore pressures, such that a liquefied deposit of clean sand will tend to re-solidify more rapidly than a silty sand. For these reasons, lateral spreading is influenced by the fines content of the liquefied soil. In scale model tests conducted by Toyota and Towhata (1994), significantly greater deformations resulted when fines were added to a clean sand. More significantly, Bartlett and Youd (1992a; 1992b; 1995) have observed a correlation between fines content and the magnitude of displacements in liquefaction-induced lateral spreads in the field.

Plastic fines within a predominately sandy soil hinder the relative movement of the coarser grains and thereby inhibit the generation of excess pore pressure during cyclic loading. In sufficiently large proportions, plastic fines create enough particle adhesion to prevent liquefaction. On the other hand, for soils with less than 30% fines, Koester (1994) has found that the total fractional weight of fines has a more pronounced effect on the liquefaction resistance, at a given void ratio, than the plasticity of the fines.

Other laboratory studies indicate that, for coarse-grained soils at a given net void ratio, the resistance to liquefaction is reduced by the addition of non-plastic fines in the soil matrix (Finn et al. 1994). Koester (1994) observed a minimum cyclic shear strength with approximately 20-30% fines, and also a significant change in the residual strength with variations in fines content. Koester concludes that saturated sand mixtures with up to about 24% fines may be inherently collapsible and thus more likely to liquefy under cyclic loading.

Note that the fines content also affects the in situ penetration resistance as measured in a Standard Penetration Test (SPT). That is, for soils with the same cyclic shear strength, the penetration resistance is lower in soils with a higher proportion of fines. Consequently, when comparing soils with the *same penetration resistance*, the cyclic shear strength is greater for soils having more fines. As pointed out by Finn and his co-authors (1994), these observations about the influence of fines content on the liquefaction resistance are not contradictory because the fines content affects both cyclic shear strength and in situ penetration resistance. This phenomena is indicated in the widely-used empirical methods for evaluating liquefaction susceptibility based on SPT data (see Chapter 7) where, for a given SPT resistance, the liquefaction resistance increases with fines content.

3.5. Movement of Pore Water in a Lateral Spread.

Upward migration of pore water.

A potentially significant phenomenon in a lateral spread is the upward flow of pore water *within* the liquefied soil deposit without a change in the overall volume (*Liquefaction...* 1985). In a liquefied state, individual soil grains tend to sink under the influence of gravity. The net effect is for the liquefied soil to progressively densify near the bottom and loosen near the top, as depicted in Figure 3.4. Stated another way, there is a tendency for an upward migration of pore water and void space. Lateral spreading may then occur as blocks of soil slide over looser, weaker soil near the top of the liquefied deposit. If the liquefied soil is overlain with a relatively impervious layer, upward flowing pore water will be trapped and contribute to the formation of a weaker soil zone in the top portion of the liquefied deposit.

Since the movement of water and soil grains takes time, Castro (1987) suggests that this phenomenon might develop in relatively thin layers but not throughout the depth of thick liquefied deposits. However, even in a thick, uniform soil deposit, the tendency for soil grains to settle would result in the formation of progressively, slightly weaker soil toward the top of the liquefied deposit. Youd and Bennett (1983) report possible field evidence for the upward migration of pore water in a liquefied deposit in a lateral spread. Below a clayey, surficial soil at the site studied by Youd and Bennett, the penetration resistance was significantly lower in the upper one to two meters of a thick sand deposit that liquefied. recent centrifuge model tests have also produced experimental evidence of this mechanism in liquefied soil deposits under surface slopes (Arulanandan et al. 1993; Fiegel and Kutter 1994a; 1994b).

Because of pore water migration, the shear resistance near the top of a liquefied deposit may be significantly lower than the shear strength corresponding to the average void ratio of the liquefied material (Seed 1987). The method proposed by Poulos et al. (1985) for determining the residual shear strength of a liquefied deposit, discussed in Section 3.4, assumes no change in void ratio during an earthquake. Because of pore water and void ratio redistribution, the actual shear strength affecting displacements in a lateral spread may be significantly lower than would be predicted based on the average void ratio of the deposit.

Drainage of excess pore pressures.

Liquefaction is generally considered to result from the generation of excess pore pressures under "undrained" conditions, or at least under conditions where shear loads are applied more rapidly than pore pressures can dissipate. Since some degree of pore water drainage almost always occurs in nature, assuming undrained conditions in the field when assessing the triggering of liquefaction is a conservative analysis approach (Seed 1979). However, the migration of pore water in partially drained conditions can have a significant impact on the magnitude of displacements in a lateral spread. Stark and Mesri (1992) note that partial drainage of excess pore pressures can produce an increase in shear resistance as sliding progresses. Rapid drainage of excess pore pressures from the liquefied soil zone may be sufficient to stabilize the slide before large deformations occur.

On the other hand, excess pore pressures generated in a liquefied soil deposit can increase the pore pressures in adjacent soils. This can reduce the shear resistance of the surrounding soils and contribute to greater slope movements. Seed (1979) points out that the migration of excess pore pressures might cause liquefaction of adjacent soils, even when the adjacent soil would not otherwise be subject to liquefaction in a given seismic event.

3.6. Deformation Within the Liquefied Deposit.

Viscous liquid analogy.

Based on shake table tests, several Japanese authors (Towhata 1991; Towhata et al. 1991; Sasaki et al. 1992, Doi and Hamada 1992) have suggested that the behavior of liquefied soil in a lateral spread is analogous to a viscous liquid. Accordingly, movement of the ground surface proceeds in the direction of decreasing total "head", defined as the total overburden pressure (Sasaki et al. 1992), just as a liquid with a sloping surface will flow until a level surface is achieved. However, unlike a viscous liquid, lateral spreading rarely ends with a level surface: deformations stop when the driving and resistive forces equilibrate, usually with some residual surface slope. Also unlike a viscous liquid, liquefied soil has a finite shear resistance that increases as excess pore pressures dissipate. Although clearly not correct in all aspects, the analogy of a viscous liquid is still helpful in understanding the mechanics of lateral spreading.

When the ground surface is initially level, no gradient exists (even if the bottom of the deposit is inclined) and the viscous liquid analogy predicts no horizontal movement. Some model tests have shown no horizontal movement when the liquefied deposit has a level surface but inclined bottom (Sasaki et al. 1992); however, other model test results show a correlation between displacement and bottom slope (Yasuda et al. 1992a). With an initially level ground surface, horizontal movements may result from differential settlements (Towhata 1991). A level surface over a liquefied deposit of variable thickness may become sloped as a result of greater settlements, due to drainage of pore water, in the thicker sections. Consequently, displacements may occur in the direction of greater thickness of liquefied soil, even when the ground surface is initially level.

Considering the liquid flow analogy for lateral spreading more closely, Doi and Hamada (1992) suggest that the velocity of surface displacements, and not the ultimate magnitude, is influenced by the pressure gradient or surface slope. Doi and Hamada use this argument to partially explain the poor correlation they found between surface slope and displacement magnitude in actual lateral spreads. Indeed, shake table models tested by Miyajima et al. (1991) have shown a correlation between velocity of movements and surface slope. However, if steeper slopes cause faster movements, then, all else being equal, greater displacements should result. The influence of surface slope is discussed further in the next section.

In recent model tests, Hamada et al. (1994) noted a sudden drop in the excess pore pressures in a liquefied deposit near the end of gravity flow. In other words, after large deformation, the effective stress increased and the shear strength of the liquefied soil was re-mobilized. This suggests a refined view of liquefaction-induced ground flow where the liquefied soil experiences two phase transformations. First, the soil liquefies during dynamic loading and flows like a viscous liquid; then, at large strains, the soil re-solidifies and flow ceases. The magnitude of shear strain at which the liquefied soil stiffness is recovered has been termed the "critical strain" by Hamada et al. (1994) or the "reference strain at resistance transformation" by Yasuda et al. (1994). Hence, while a liquefied deposit may flow like a viscous liquid, deformations stop when the liquefied soil re-solidifies at large strains prior to reaching a level surface.

Deformation of vertical sections.

Scale model simulations of lateral spreading consistently demonstrate that sliding in a lateral spread does not occur along a well defined failure surface; that is, surficial blocks do not simply slide over the top of the liquefied deposit. Instead, as depicted in Figure 3.5a, down-slope movements of surficial soil blocks are associated with shear distortions across the thickness of the liquefied soil deposit (Towhata 1991; Sasaki et al. 1991; 1992; Yasuda et al. 1992a; Fiegel and Kutter 1994b; Elgamal et al. 1996). Horizontal displacements are greater in the upper portion of the liquefied deposit and decrease with depth, becoming negligible at the bottom of the liquefied deposit. In this respect, the liquefied soil is seen to behave like a very soft solid or viscous fluid.

This distribution of horizontal displacements across the thickness of the liquefied deposit has also been observed in the field. Foundation pilings and buried pipes bent and damaged in lateral spreads in Niigata, Japan, and subsequently excavated indicated that the entire thickness of the liquefied deposit was moving down-slope (Doi and Hamada 1992). At the Wildlife Liquefaction Array, which experienced lateral spreading in 1987, the curvature of an inclinometer casing showed greater shear strains in the upper portion of the liquefied deposit (Holzer et al. 1989). Three inclinometer casings subjected to lateral spreading in 1989 at Moss Landing, California, indicated a distribution of lateral displacements consistent with that shown in Figure 3.5 (Barminski 1993; Boulanger et al. 1997). The data from Moss Landing suggest that:

> *"deformations developed as relatively uniform shear strains over the thickness of the strata that liquefied or were softened . . . and not as deformations concentrated along a single thin zone or failure plane"* (Boulanger et al. 1997).

In centrifuge models of lateral spreading, Fiegel and Kutter (1994b) studied the affect of an impervious surface layer on top of a liquefied soil. When the liquefied soil extends to the surface, a curved lateral displacement profile is typically observed as shown in Figure 3.5a. Fiegel and Kutter believe this behavior is the result of post-liquefaction solidification which progresses from the bottom upward. That is, after liquefying, the soil grains tend to settle toward the bottom as pore water migrates upward. Consequently, the surficial soil is in a liquefied state for a longer period of time and greater horizontal displacements occur in the upper portions of the deposit. On the other hand, when the liquefied soil is overlain by a relatively impervious soil, greater deformations are observed in a narrower band just beneath the interface as depicted in Figure 3.5b. This can be explained as resulting from the accumulation of upward flowing pore water trapped below the interface with the surficial soil. Thus, the displacement profiles in model tests can be explained by the phenomena of upward migration of pore water in the liquefied deposit, as discussed earlier in Section 3.5.

Influence of liquefied thickness.

Numerous researchers have consistently observed a correlation between horizontal surface displacements in a lateral spread and the thickness of the underlying liquefied soil deposit. This correlation has been observed in laboratory scale models (Yasuda et al. 1992a; Tokida et al. 1993) and in statistical studies of lateral spreading in the field (Hamada et al. 1986; 1987; Bartlett and Youd 1992a; 1992b; 1995; Doi and Hamada 1992; O'Rourke and Pease 1997). The influence of the liquefied thickness might be explained with the following mechanisms:

(1) Since shear deformations occur across the full depth of the liquefied deposit, the net surface displacement will increase with greater liquefied thickness. If the shear strain was constant with depth, the lateral displacement profile would be linear and, for a given shear strain, the net surface displacement would increase with liquefied thickness. Of course, shear strains may vary with depth in a lateral spread consistent with the displacement profiles depicted in Figure 3.5.

(2) As the thickness of the liquefied soil deposit increases, a greater upward migration of pore water might occur. Then, the significance of the resulting soft zone near the top of the deposit would increase with an increasing liquefied thickness.

(3) The drainage path for excess pore pressures increases with the thickness of the liquefied soil. As a result, portions of a thicker liquefied deposit will remain liquefied for a longer period of time and cause greater surface displacements.

3.7. Boundary Effects.

Influence of surface slope.

Youd and Kiehl (1996) observed that the direction of lateral spreading movement is generally determined by the surface slope. More importantly, greater displacements are intuitively expected in lateral spreads with steeper surface slopes. A steeper slope imparts greater static shear forces and, based on the viscous liquid analogy, results in a faster deformation velocity. Shake table model studies have shown that displacements increase with surface slope (Sasaki et al. 1991; 1992; Miyajima 1991). However, statistical studies of lateral spreading in the field have shown a poor correlation between displacement magnitude and surface slope (Hamada et al. 1986; 1987; Doi and Hamada 1992, Hamada 1992a; 1992b; O'Rourke and Pease 1997), although a significant correlation was found by Bartlett and Youd (1992a; 1992b; 1995). This lack of statistical correlation, however, may be related to the small range of fairly gentle surface slopes represented in the field case histories and may not be indicative of the physical mechanisms involved.

These seemingly conflicting conclusions between laboratory and field studies may also result from the difficulty in defining surface slope. In nearly all scale model tests, uniform slopes are constructed and measurement of the surface slope is simple. In the natural world, however, even fairly flat ground will always include some undulations, hummocks, and small depressions. Under these conditions, it is not easy to define a representative surface slope, and it is difficult to judge the relative importance of small topographic features or even the local direction of maximum slope. As pointed out by S. F. Bartlett (personal communication, 1994), defining surface slope as the change in elevation over some fixed horizontal distance is bound to exclude some important topographic features. For example, consider the movement of a reference point on ground at a given local slope. Greater displacement would be expected if this point was near a steep embankment than if the point was located on a long, uniform slope at the same surface gradient. To include consideration of important topographic features, Bartlett and Youd (1992a; 1992b) used a variable length to define surface slope at a point, as discussed in Section 4.4.

In reality, the movement of specific points on a lateral spread may be related to two different measures of surface slope. First, the average slope across the entire slide area will affect the average movement of the slide mass. Secondly, local topographic features on the slide mass, such as small hummocks or depressions, may alter the displacements from point to point on a lateral spread. That is, displacements at specific locations may differ somewhat from the general trend of surface movements due to variations in local slope. Unquestionably, surface topography plays a key role in the movements of lateral spreads, although the influence of surface slope is difficult to quantify.

Slide boundaries.

The magnitude and direction of displacements in a lateral spread may be affected by natural or constructed features in the vicinity. For instance, O'Rourke and Lane (1989) report that a lateral spread in San Francisco was strongly influenced by the underlying topography. In this particular case, the flow direction changed as the slide followed the course of a buried ravine and, where the buried ravine narrowed, greater surface movements were observed in the relatively narrower zone. A very common natural feature is a steep free face at the toe of a slide, such as a stream bank, that provides an unrestricted boundary leading to greater horizontal displacements (Bartlett and Youd 1992a; 1992b; 1995; Youd and Kiehl 1996). On the other hand, retaining walls, tunnels, pipelines, and other civil works can constrict the movement of a slide as evidenced in another lateral spread in the city of San Francisco (Clough et al. 1994).

The boundaries of a lateral spread are usually defined by the general limits of the liquefied deposit. These limits are sometimes defined by the contact between different geologic units at the same depth (for an example, see Holzer et al. 1994). When a loose, sandy deposit is found above the water table over part of a site, the boundary of a lateral spread may coincide with the saturated edge of the deposit. Ground surface failures can also extend for some distance beyond the edges of the liquefied deposit by a mechanism identified as *marginal slumping* (O'Rourke and Lane 1989). Marginal slumping occurs when movement of soil blocks in a lateral spread releases the lateral support to soil further up the slope. Ground deformations may then occur beyond the zone of the liquefied soil, as depicted in Figure 3.6. Marginal slumping has been identified in lateral spreads in San Francisco (O'Rourke et al. 1992a) and San Fernando, California (O'Rourke et al. 1992b).

3.8. Inertial Effects.

Inertial and static stresses.

In a liquefaction-induced slope failure, sliding occurs when the net driving forces exceed the net resisting forces on the slide mass, both during and after the earthquake. Resistance to sliding is provided by intact soils, along the sides and toe of the slide, and the shear resistance of the underlying, liquefied soil deposit. During an earthquake, slope movements are driven by both inertial and static forces. After the seismic motions stop, slope deformation may continue under the action of static shear stresses if sufficient shear resistance is lost. In this context, static driving shear loads are equal to the forces required to maintain static equilibrium in the sloping ground mass (Castro 1987).

In a steep embankment, high static shear stresses may cause tremendous displacements before equilibrium is re-established with the reduced shear resistance of a liquefied deposit. However, in a lateral spread of gently sloping ground, the static driving shear stresses are relatively low such that more modest ground deformations would be expected. On the other hand, low static driving shear is more likely to result in a reversal of the shear stress direction during cyclic loading leading to a temporary, zero effective stress condition (Robertson and Fear 1996), a phenomenon described earlier in Section 2.1. Consequently, low static driving shear in a gentle slope may contribute to somewhat greater displacements during an earthquake while limiting deformations after seismic motions stop.

In an effort to assess the relative influence of static and inertial forces on a lateral spread, Sasaki et al. (1991; 1992) conducted a special model test on a shake table. In this test, a conical embankment was liquefied with unidirectional base motions. The resulting slope movements were generally radial, and were not significantly greater in the direction of shaking. The results of this test suggest that static shear loads predominate over inertial shear loads (Sasaki et al. 1992; Towhata 1991), although this conclusion may not hold for all combinations of slope and shaking intensity. For example, Fiegel and Kutter (1994b) observed greater displacements occurring during shaking in centrifuge model tests indicating that inertial forces are significant in a lateral spread of gently sloping ground.

Ground oscillations.

Another type of liquefaction failure closely related to lateral spreading is *ground oscillation* (Youd and Garris 1994; 1995). On level ground, liquefaction of an extensive, subsurface deposit may result in a back and forth movement (oscillation) of intact ground over the liquefied soil as the surface displacements de-couple from the underlying seismic motions. Pease and O'Rourke (1997) explain ground oscillation failures in terms of greater surface movements resulting from amplification of low-frequency seismic motions when transmitted through a liquefied soil deposit. Using numerical simulations and records of ground motions during the 1989 Loma Prieta earthquake, Pease and O'Rourke were able to attribute buried pipeline damage in the Marina District of San Francisco to low-frequency, transient displacements that were amplified by the underlying liquefied soil. The resulting surface displacement waves, with periods of 4 to 6 sec, caused ground deformation failures that were identified as ground oscillations.

A simple ground oscillation failure may buckle or pull apart linear, rigid elements (such as rails and curbs) without generating a consistent pattern of ground displacements. On the other hand, a lateral spread will produce movements in a fairly consistent direction that distinguish the slide mass. However, lateral spreads often include some ground oscillation, which cause displacements that are inconsistent with the general deformation pattern. For example, in a lateral spread in San Francisco, compressive deformations were observed in an area where tensile failures would be expected, a feature attributable to ground oscillations (O'Rourke et al. 1992a).

Youd and Garris (1994; 1995) point out that neither ground oscillation nor lateral spreading is likely to occur unless the liquefied deposit covers an area broad enough to allow decoupling of the surficial soils from the solid ground beneath the liquefied deposit. Isolated lenses of liquefied soil are not likely to result in surface disruptions from ground oscillations or lateral spreading.

Movements after the earthquake.

As pointed out above, the static driving shear stresses in a lateral spread are relatively low and are likely to be less than the residual shear strength of the liquefied deposit. Thus, Castro (1987) suggests that lateral spreading of a gentle slope is not likely to continue after earthquake motions cease. However, other factors may contribute to cause slope deformations that continue after an earthquake. As liquefaction and lateral spreading progresses, a redistribution of the static driving loads may result in a progressive failure. As the soil in a given location softens, forces formerly supported are transferred to surrounding parts of the slope. Liquefaction may then begin in one location and progress into adjacent areas of the same soil layer. As observed by Gu et al. (1994), this mechanism of shear stress redistribution takes time and may cause additional slope movements after earthquake shaking stops.

Eyewitness accounts of lateral spreads in past earthquakes confirm that liquefaction-induced lateral spreading movements may continue for some time after the earthquake. In Fukui, Japan, ground fissures caused by liquefaction and lateral spreading were reported to continue increasing in width for several hours after the 1948 earthquake (Hamada et al. 1992). In the Niigata, Japan, earthquake of 1964, the Showa Bridge collapsed after the cessation of earthquake ground motions in a failure attributed to lateral spreading along the banks of the river (Hamada et al. 1986; Hamada 1992a). In California in 1989, a woman standing on a lateral spread along the Pajaro River reported that ground fissures and movements occurred after the earthquake shaking had subsided (Seed et al. 1990).

Based on model tests and field observations, one can thus conclude that both static and inertial forces play some role in driving the down-slope movements in a lateral spread, and such displacements may continue for some time after the seismic ground motions stop.

3.9. Settlements.

Most damage in a lateral spread results from large horizontal displacements. Vertical displacements, including both settlements and heaving, are usually less significant on a lateral spread but can cause considerable damage. Heaving typically occurs at the toe of a slide, while substantial settlements can occur at any location. Vertical displacements are often coupled with the horizontal movement and tilting of unbroken blocks of surficial soil. Settlements result from the ejection of material in sand boils and reconsolidation of liquefied deposits. In addition, structures on shallow supports may experience substantial or catastrophic settlements due to bearing capacity failures into the liquefied foundation soils.

Using data from sites in San Francisco subjected to liquefaction in 1989, O'Rourke and Pease (1997) have shown a fairly strong linear relationship between liquefied thickness and settlements observed in several lateral spreads. The exact correlation between settlement and liquefied thickness varied among the sites studied: these differences were interpreted by O'Rourke and Pease to indicate the volumetric strain of the liquefied soil at each site. Liquefaction-induced settlement of level ground can be predicted based on the thickness and potential volume strain of the liquefied soil deposit using the methods described in Section 4.5.

3.10. Summary.

Tremendous damage in large-magnitude earthquakes often results from soil liquefaction and lateral spreading. Lateral spreading is defined here as the movement of soil blocks on gently sloping ground over shallow, liquefied soil deposits. In an attempt to consolidate the current understanding of lateral spreading behavior, the significant aspects of the physical processes involved have been reviewed in this chapter.

Various aspects of liquefied soil behavior play an important role in the mechanics of a lateral spread. The fines content of a soil deposit affects both the liquefaction characteristics and the rate of pore water drainage and thus affects the magnitude of displacements in a lateral spread. In addition, relatively low confining pressures at shallow depths may result in a dilative response in loose, liquefiable soils. Coupled with the low static shear stresses in a gentle slope, this suggests that lateral spreading might not occur at the undrained, residual shear strength often associated with liquefaction failures. The residual shear strength is difficult to estimate for any liquefiable soil, and this estimate is further complicated by the movement of pore water and the possible formation of a significantly weaker soil zone near the top of a liquefied deposit. Hence, the residual shear strength of a liquefied deposit is probably not a strong predictor of displacement magnitudes in a lateral spread.

The analogy of a viscous liquid can be useful in understanding the deformation of the liquefied soil deposit in a lateral spread. Surficial blocks of soil do not slide along a well-defined failure surface or over the top of the liquefied deposit. Instead, laboratory models and field evidence indicate that horizontal deformations vary across the depth of the liquefied deposit from zero at the bottom to a maximum near the top. Consequently, the magnitude of displacements can be correlated to the thickness of liquefied soil.

Conditions along the sides of a lateral spread also affect the magnitude of displacements by constricting or resisting movement of the slide mass. A vertical face at the toe of the slide, such as a stream bank, allows for greater horizontal movements. Slumping along the margins of a lateral spread can effectively extend the area of ground failure beyond the limits of the liquefied deposit. While it seems that surface topography should significantly influence the ultimate movement of a lateral spread, poor correlations are often found between surface slope and displacement magnitudes. However, this may result from the difficulty in precisely quantifying the surface slope on relatively level ground while considering both the average slope and localized topographic features across a slide area.

Finally, both static forces due to the slope and seismic, inertial forces contribute to the deformation of a lateral spread. Inertial forces can cause back and forth movements in an oscillation failure of level ground; evidence of these motions are frequently observed in lateral spreads. However, once liquefaction has been triggered in an earthquake, the small static forces in a gentle slope can be sufficient to cause movement. Eyewitness accounts of lateral spreading have indicated that significant displacements can occur under static, gravity stresses after the cessation of seismic motions.

Liquefaction-induced lateral spreading can be seen to involve a complicated interaction of many factors. These factors have significant implications in choosing methods to model lateral spreading and predict ultimate displacements, as discussed in Chapter 4.

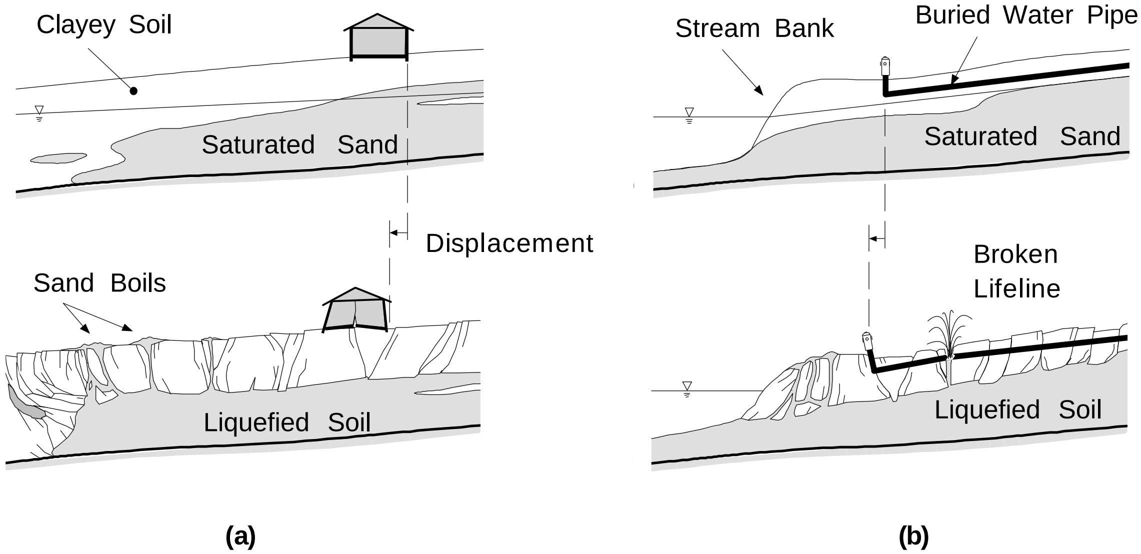

Figure 3.1. Soil liquefaction and lateral spreading of (a) gently sloping ground and (b) toward a free face.

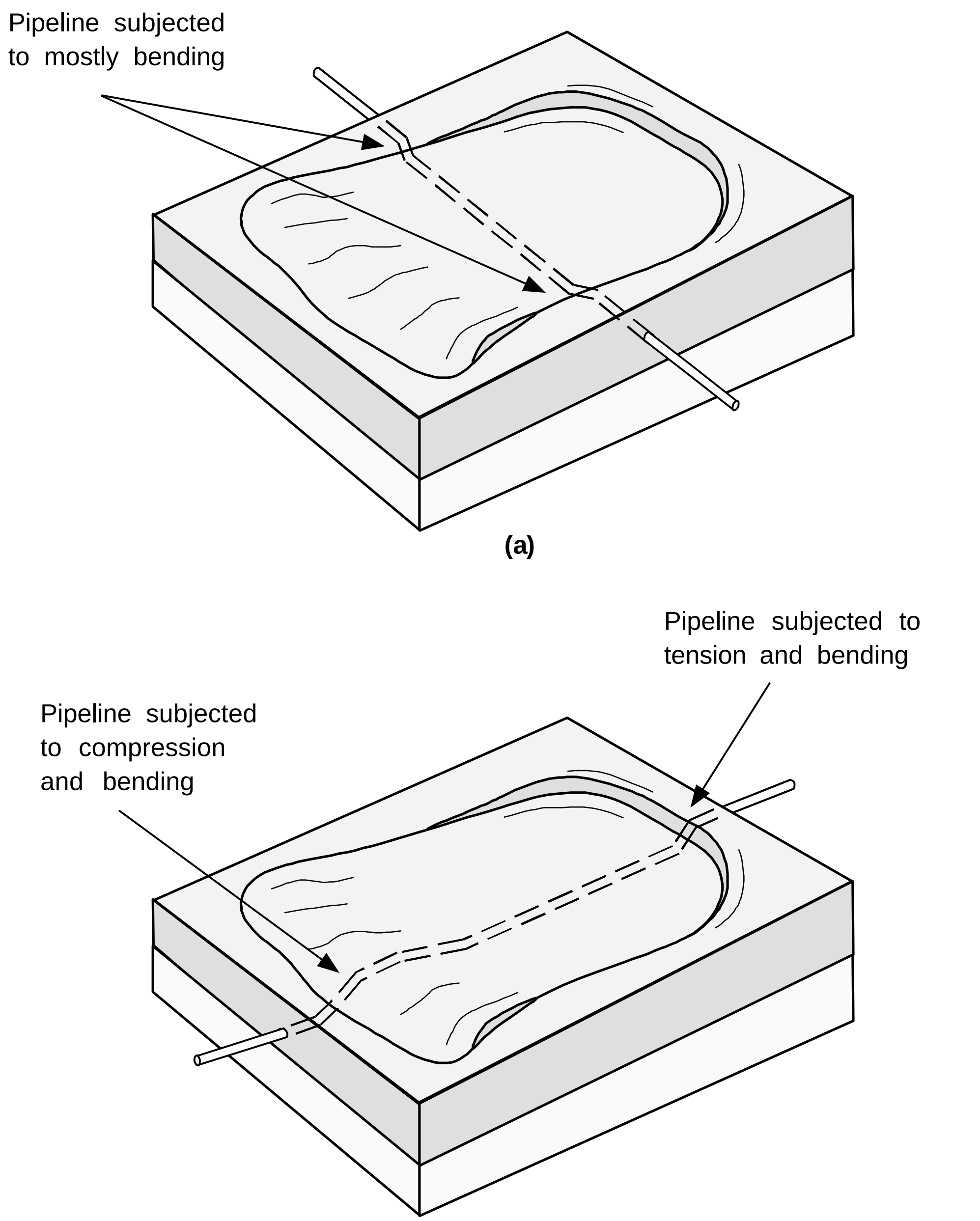

Figure 3.2. Pipeline damage in (a) perpendicular and (b) parallel crossings of a lateral spread (after O'Rourke and Lane 1989).

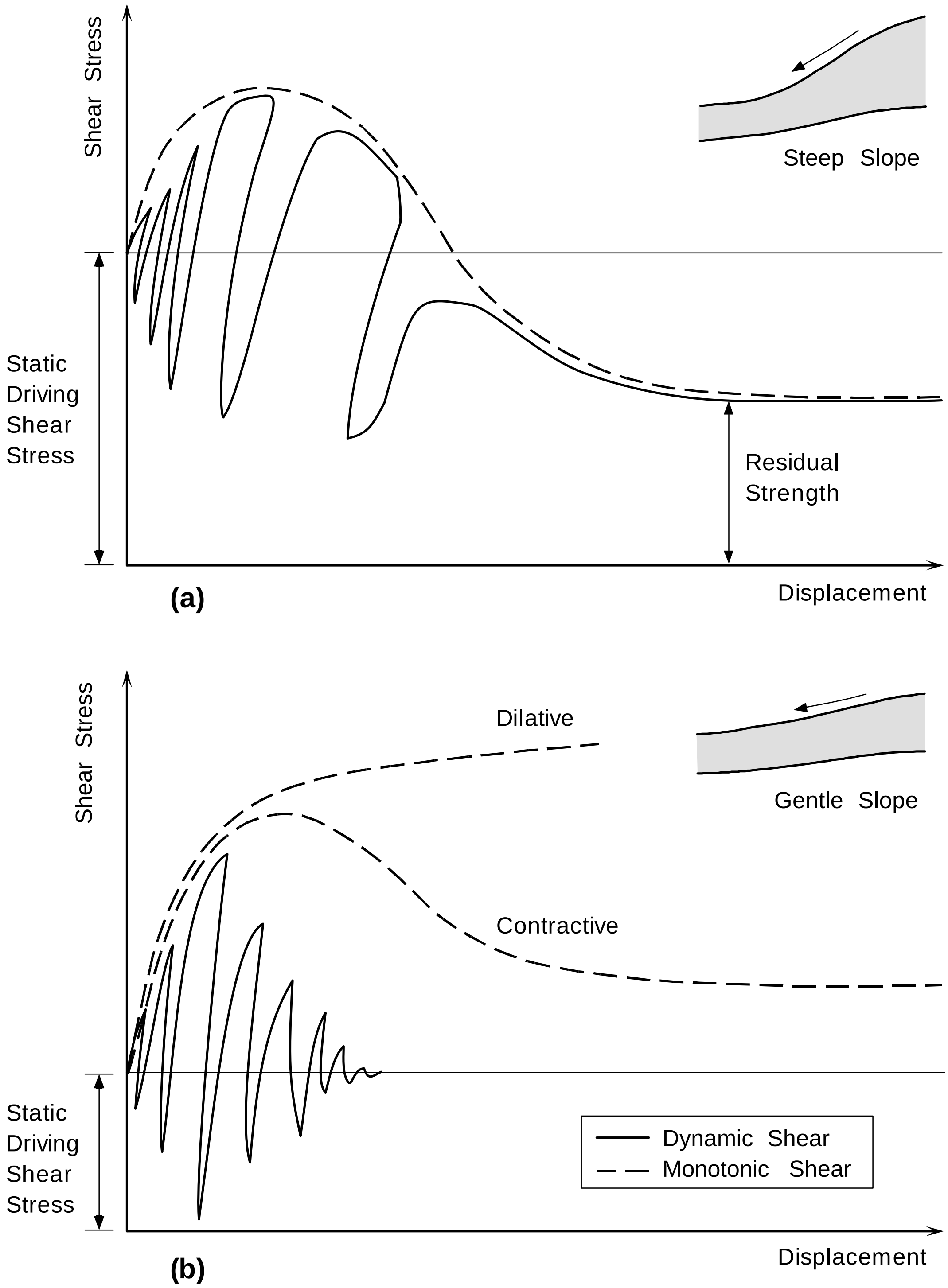

Figure 3.3. Earthquake-induced slope movements: (a) flow failure in a contractive soil, and (b) limited deformation in a dilative or contractive soil (after Ishihara 1994).

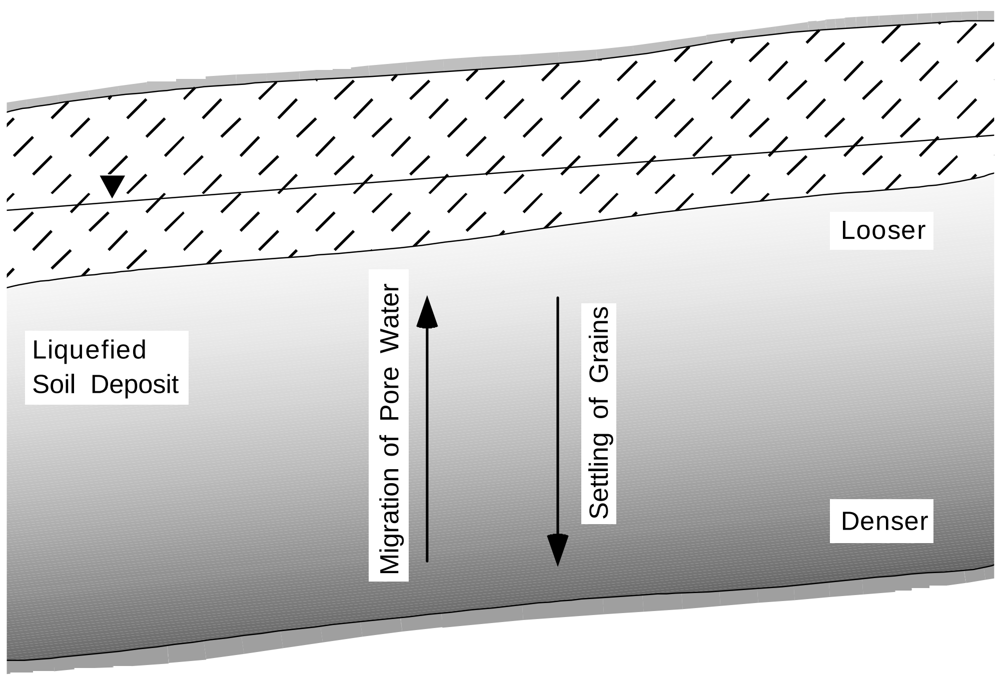

Figure 3.4. Settling of soil grains in a liquefied soil deposit leading to the upward migration of pore water (after *Liquefaction...* 1985).

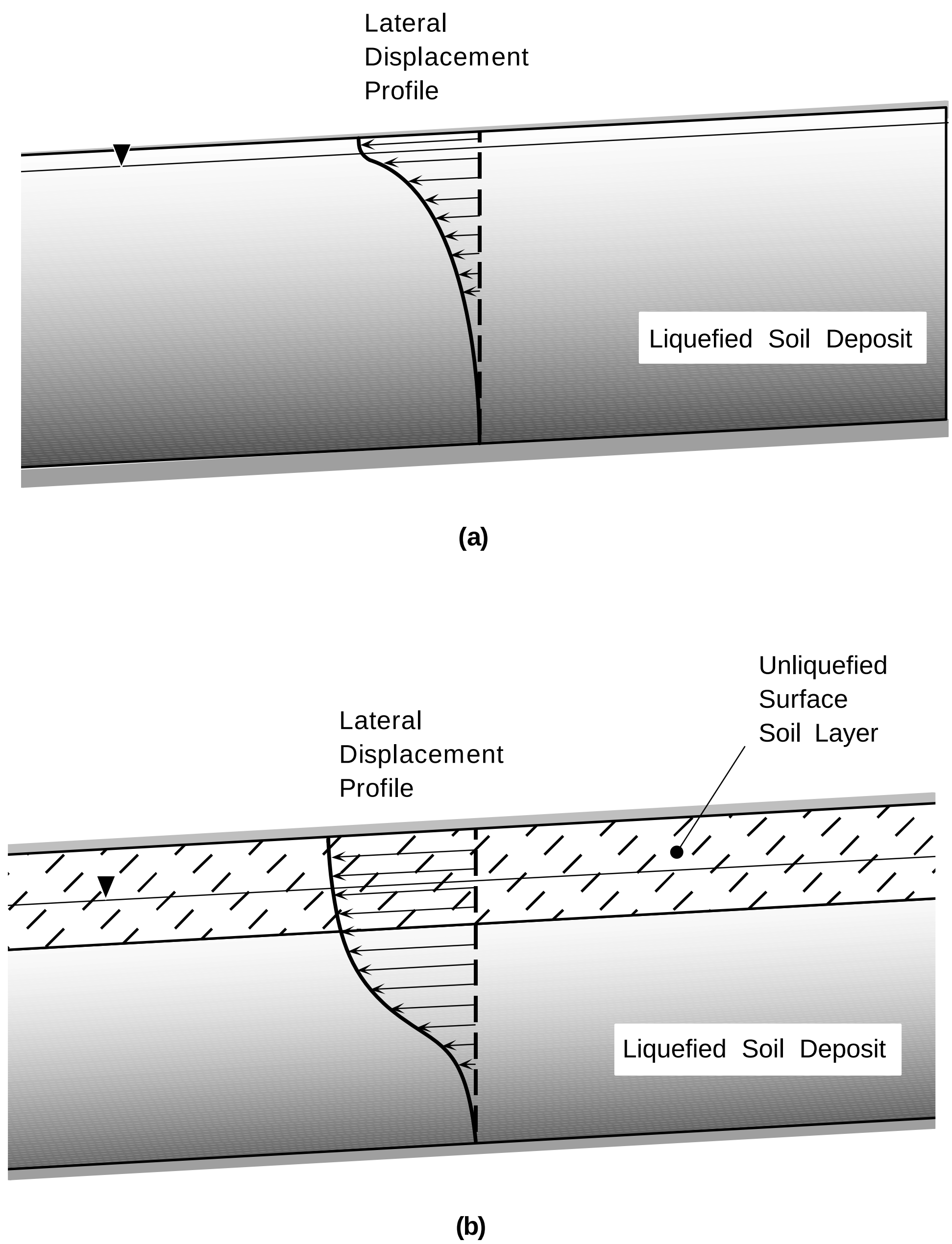

Figure 3.5. Lateral displacement of vertical sections within the liquefied deposit of a lateral spread with a surface layer that is (a) free draining or (b) impervious.

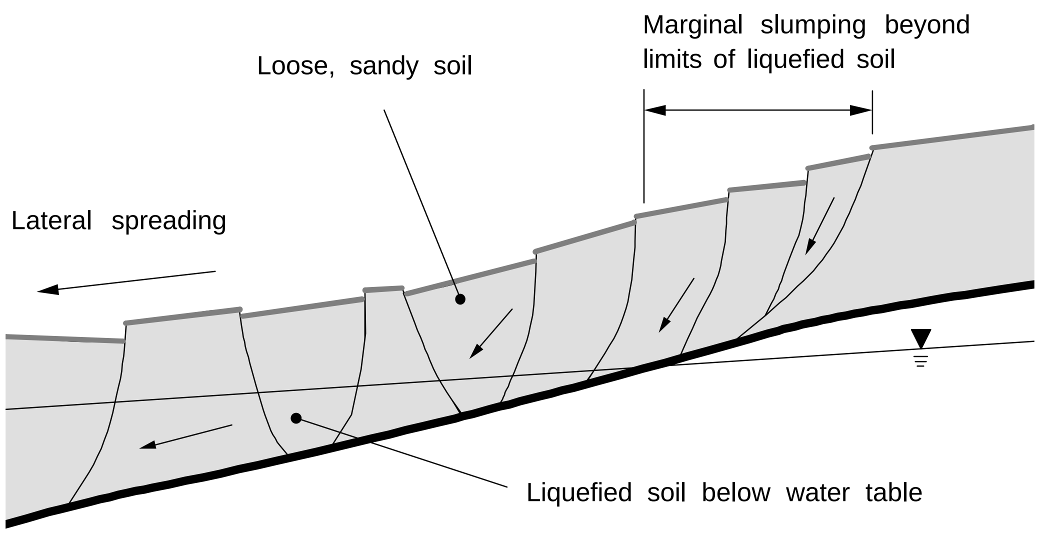

Figure 3.6. Marginal slumping around a lateral spread (after O'Rourke and Lane 1989).