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# Liquefaction, Flow, and Associated Ground Failure

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By T. Leslie Youd

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United States Department of the Interior
ROGERS C. B. MORTON, *Secretary*

Geological Survey
V. E. McKelvey, *Director*

*Free on application to the U.S. Geological Survey, Washington, D.C. 20244*

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ABSTRACT

Ambiguities in the use of the term liquefaction and in defining the relation between liquefaction and ground failure have led to encumbered communication between workers in various fields and between specialists in the same field, and the possibility that evaluations of liquefaction potential could be misinterpreted or misapplied. Explicit definitions of liquefaction and related concepts are proposed herein. These definitions, based on observed laboratory behavior, are then used to clarify the relation between liquefaction and ground failure.

Soil liquefaction is defined as the transformation of a granular material from a solid into a liquefied state as a consequence of increased pore-water pressures. This definition avoids confusion between liquefaction and possible flow-failure conditions after liquefaction. Flow-failure conditions are divided into two types: (1) unlimited flow if pore-pressure reductions caused by dilatancy during flow deformation are not sufficient to solidify the material and thus arrest flow, and (2) limited flow if they are sufficient to solidify the material after a finite deformation. After liquefaction in the field, unlimited flow commonly leads to flow landslides, whereas limited flow leads at most to lateral-spreading landslides. Quick-condition failures such as loss of bearing capacity form a third type of ground failure associated with liquefaction.

## INTRODUCTION

Because of the critical effect liquefaction has on the safe performance of engineered construction and the stability of certain geologic formations, considerable study has been devoted to this topic in recent years. Significant progress has been made in understanding the liquefaction phenomenon and the factors controlling it; however, ambiguity in present definitions of the term liquefaction and lack of clear distinction between liquefaction and ground-failure conditions associated with this phenomenon have produced some confusion in usage and understanding of liquefaction and its consequences. This confusion has encumbered communication between workers in various disciplines as well as between specialists in the same field and allowed the possibility that analyses of liquefaction potential could be misinterpreted or misapplied.

In this paper explicit definitions of liquefaction and related concepts are given on the basis of behavior observed in laboratory tests. These concepts are then used to define relations between liquefaction and the various types of ground failure commonly associated with liquefaction. Examples are given to illustrate the character of each type of ground failure. Finally, the applicability and limitations of existing methods for evaluating liquefaction potential are briefly examined with the aid of the definitions formulated herein.

Appreciation is given to D. H. Gray, E. J. Helley, Kaare Hoeg, K. L. Lee, H. W. Olsen, and H. B. Seed for comments and suggestions.

## PUBLISHED DEFINITIONS OF LIQUEFACTION

The standard technical dictionary definition of liquefaction is "The act or process of transforming any substance into a liquid" (Lange and Forker, 1961, p. 1738). Several definitions of soil liquefaction are in basic agreement with this concept. For example, Terzaghi and Peck (1948, p. 100) defined spontaneous liquefaction as "the sudden decrease of shearing resistance of a quick sand from its normal value to almost zero without the aid of seepage pressure." A generally comparable definition was published by the American Society of Civil Engineers (1958, p. 1826–22) and quoted by the American Geological Institute (1972, p. 410) as follows: "The sudden large decrease of shearing resistance of a cohesionless soil, caused by a collapse of the structure by shock or strain, and associated with a sudden but temporary increase of the pore fluid pressure [is liquefaction]. It involves a temporary transformation of the material into a fluid mass." Ghaboussi and Wilson (1973) gave the following definition: "The phenomenon of the loss of strength of saturated granular soils during earthquakes is generally referred to as liquefaction. The process of liquefaction transforms an element of soil from a state of saturated granular solid to a state of viscous fluid."

Several definitions for liquefaction and related phenomena have been proposed by Seed and his coworkers on the basis of behavior observed in cyclically loaded shear tests on saturated samples of sand. Lee and Seed (1967a, p. 49) define (1) complete liquefaction as "when a soil exhibits no resistance (or negligible resistance) to deformation over a wide strain range, say a double amplitude of 20 percent"; (2) partial liquefaction as "when a soil exhibits no resistance to deformation over a strain range less than that considered to constitute failure"; and (3) initial liquefaction as "when a soil exhibits any degree of partial liquefaction during cyclic loading." In addition Seed and Idriss (1971, p. 1249) have used the term liquefaction to describe "a phenomenon in which a cohesionless soil loses strength during an earthquake and acquires a degree of mobility sufficient to permit movements ranging from several feet to several thousand feet." Except for initial liquefaction, both a flow deformation requirement and a strength loss requirement are incorporated into each of the definitions proposed by Seed and his colleagues.

Castro (1969, p. 7) used the term liquefaction to denote the following phenomenon: "The conventional use of the term [liquefaction] as it will be used throughout this thesis, refers to the phenomenon which takes place in a mass of soil during flow slides. Liquefaction or flow failure of a sand is caused by a substantial reduction of its shear strength." This usage in effect defines liquefaction as a condition of unrestrained flow deformation.

The incorporation or confusion of two different phenomena (strength loss leading to a material phase transformation and flow deformation) into a single definition has led to ambiguities in usage. For example, a map of sand deposits susceptible to liquefaction could be interpreted as (1) a map of deposits that could be transformed into a liquefied state, but with no assessment of type or amount of ground-failure movement, if any, that might follow the transformation; (2) a map of deposits that could lose strength and acquire a degree of mobility sufficient to permit movements ranging from several feet to several thousand feet; or (3) as a map of deposits that could lose strength and fail in the form of flow landslides. In order to reduce this ambiguity and provide for more precise usage, explicit definitions are needed, and the relation between liquefaction and ground failure needs clarification.

## OBSERVED BEHAVIOR AND PROPOSED DEFINITIONS

Laboratory test data illustrative of the behavior of saturated, undrained granular soils during shear loading (monotonic and repetitive) have been published by several investigators (Castro, 1969; Finn and others, 1970; Lee, 1970; Lee and Fitton, 1968; Lee and Seed, 1967a, b; Peacock and Seed, 1968; Seed and Lee, 1966; 1969). These data are used here as a basis for formulating definitions of liquefaction and related terms, clarifying the relation between liquefaction and ground failure, and examining the character of various types of ground failure associated with liquefaction.

Some pertinent aspects of the behavior of saturated sands during conditions of undrained, monotonic shear loading are illustrated in the curves plotted in figure 1. These data were taken from test 4–7 of Castro (1969), a saturated undrained triaxial compression test on moderately dense (relative density; $D_r$=47 percent[1]) Ottawa sand. Three distinct phases, through which the sample passed, are shown on the curves. (1) During initial loading (beginning of test to point *l*), the sample behaved as a solid (did not perceptibly flow (Lange and Forker, 1961, p. 1794)). During this part of the test, pore pressures increased with applied load until a point of instability was reached at point *l*. This rise in pore pressure was caused partly by the tendency of all but very dense samples to compact initially during loading (Youd, 1972a, p. 717). (2) Be-

[1]Relative density, $D_r$, in percent is defined by the following relationship:

$$D_r=\frac{e_{max}-e}{e_{max}-e_{min}}(100)$$

where $e_{max}$ and $e_{min}$ are void ratios of a given sand in its loosest and densest states, respectively, and $e$ is the void ratio of the sand at the density in question.

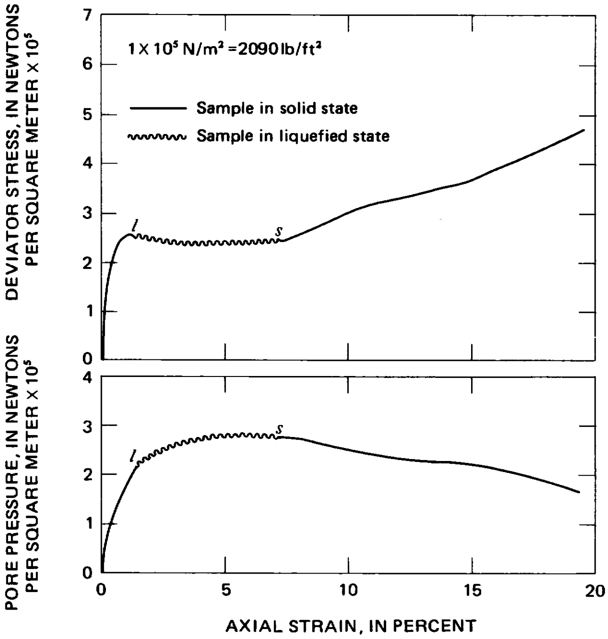


FIGURE 1.—Stress-strain and pore pressure-strain curves for monotonically loaded triaxial compression test on undrained sample of Ottawa sand illustrating liquefaction, flow deformation, and solidification (after test 4–7 Castro, 1969).

yond the point of instability (point *l*), the sample flowed in a liquefied state (rapidly deformed under constant total stress). In this phase the sample deformed through an axial strain of 6 percent in 1.0 sec. (wavy part of curves). During the latter part of the flow deformation segment, dilatant tendencies within the sample caused a reduction of pore pressure and concomitant strengthening of the sample. This eventually led to an arrest of the flow, and the sample was reconverted into a solid state (point *s* on the curves). (3) Beyond point *s* the sample continued to behave as a solid. As additional load was applied, the dilitant tendency continued to cause further pore pressure reductions and greater sample strength.

On the basis of the behavior illustrated in test 4–7 (fig. 1), the following definitions for liquefaction and related terms are proposed. *Liquefaction* is defined as the transformation of a granular material from a solid state into a liquefied state as a consequence of increased pore-water pressures. This transformation occurred during the initial loading phase of test 4–7 and was complete (liquefaction occurred) at point *l* on the curves. The proposed definition of liquefaction is in close agreement with the standard technical definition (Lange and Forker, 1961, p. 1738), the definitions of Terzaghi and Peck (1948, p. 100), the American Society of Civil Engineers (1958, p. 1826–22), the American Geological Institute (1972, p. 410), and Ghaboussi and Wilson (1973). For cyclic loading conditions, the definition proposed above is congruous with the definition of initial liquefaction given by Lee and Seed (1967a, p. 49).

At the conclusion of the liquefied flow segment of the test 4–7 (point *s* on the curves), a process opposite to liquefaction occurred—the transformation of a granular material from a liquefied state into a solid state. This process is defined here as *solidification.* Deformations in the solid state are generally referred to as *elastic strain* or *plastic strain* depending on whether deformations are recoverable or not, respectively. Deformations in the liquefied state are referred to as *flow deformations.*

Flow deformation can be divided into two types. (1) *Limited flow* (equivalent to Castro's (1969, p. 19) limited liquefaction or, for cyclic loading conditions, Lee and Seed's (1967a, p. 49) partial liquefaction) refers to a condition in which liquefaction occurs and flow deformation ensues but is arrested by solidification after a finite movement without significant change of total stresses in the process. The sample in test 4–7 (fig. 1) underwent limited flow. (2) *Unlimited flow* (equivalent to Castro's (1969, p. 16) liquefaction failure or, in essence, equivalent to Lee and Seed's (1967a, p. 49) complete liquefaction) refers to a condition in which dilatancy-caused reductions in pore pressure are insufficient to arrest flow; thus, flow deformation continues unabated until the applied shear stresses are reduced to a level less than the viscous shear resistance of the liquefied material. At that point solidification occurs. The sample in test 4–4 (fig. 2) underwent liquefaction and unlimited flow. In this test, flow deformation ceased only when the deviator stress was relieved by stopblocks in the loading apparatus.

Additional insight into the behavior of saturated sands during undrained shear can be gained from an examination of stress paths constructed from published laboratory test data. For example, effective stress paths for the three triaxial compression tests of Castro (1969) listed in figure 2

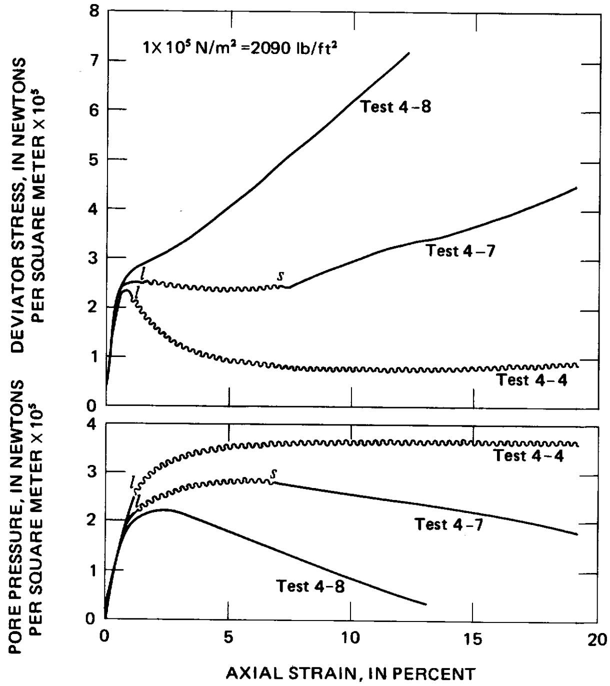


FIGURE 2.—Typical stress-strain curves for three monotonically loaded triaxial compression tests on undrained samples of Ottawa sand (after Castro, 1969). The sample in test 4–4 underwent liquefaction and unlimited flow; the sample in test 4–7 underwent liquefaction, limited flow, and solidification; and the sample in test 4–8 did not liquefy nor flow.

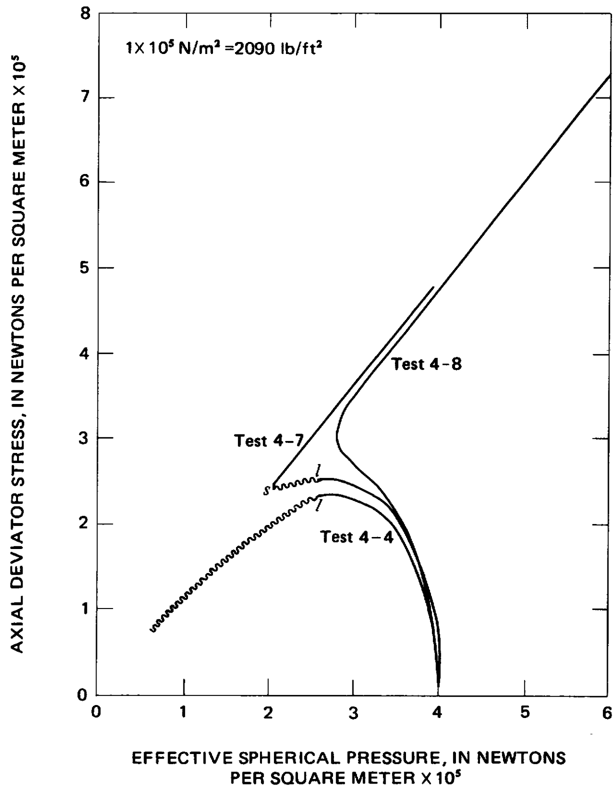


FIGURE 3.—Stress paths for the three triaxial compression tests plotted in figure 2.

have been plotted on the *p-q* diagrams (Schofield and Wroth, 1968) in figure 3,
where

$$p = \tfrac{1}{3}(\sigma_1 + 2\sigma_3) - u$$
$$q = (\sigma_1 - \sigma_3)$$

*p* is the effective spherical pressure, *q* is the axial deviator stress, $\sigma_1$ and $\sigma_3$ are the major and minor principal stresses, respectively, and *u* is the pore water pressure. In each of these tests pore pressures rose in response to the initial increments of loading. These pressures caused the stress paths to follow a counterclockwise, concave downward (strain-softening) path. The paths for tests 4–4 and 4–7 passed through a *q*-maximum (point *l* on the curves) that was also a point of instability or, as discussed earlier, the point at which liquefaction occurred. Beyond these points flow occurred (wavy parts of curves). The path for test 4–4 ($D_r$=37 percent), which underwent unlimited flow, continued downward until the end of the test. The path for test 4–7 ($D_r$=46 percent), however, passed through a *q*-minimum and then turned upward along a constant *q/p* ratio (strain-hardening) path in response to reduced pore pressures caused by dilatant tendencies within the sample. Solidification occurred at point *s* on that curve, and beyond that point the sample behaved as a solid.

The dilatant tendency became effective in test 4–8 ($D_r$=47 percent) before the stress path could read a *q*-maximum (figs. 2 and 3). Pore pressure decreased, and the stress path passed through an inflection point, became concave upward, and eventually followed a constant *q/p* ratio line congruous with the one generated during the latter part of test 4–7. Thus, neither liquefaction nor flow occurred during this test.

Although not the first to do so (Seed and Lee, 1966), Castro (1969) also demonstrated the generation of liquefaction by cyclic loading. Stress paths from two of those tests are plotted in figure 4. During the first loading, stresses followed paths similar to those of the monotonically loaded tests described above; however, loading was stopped short of a point of instability or inflection and then reduced to zero. The net result of the cycle

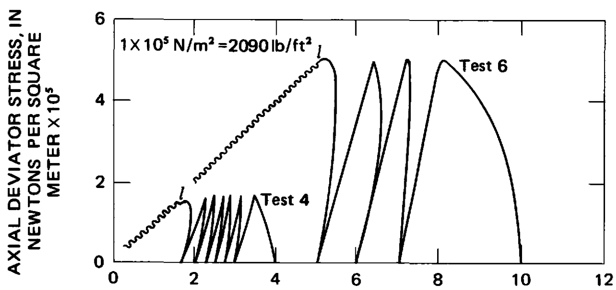


FIGURE 4.—Liquefaction and unlimited flow generated by cyclic loading. Stress paths from cyclically loaded triaxial compression tests without stress reversals. (Data from Castro, 1969.)

was a pore-pressure increase equivalent to the net decrease in $p$. Similar pore-pressure increases were generated during each succeeding cycle until liquefaction occurred on the seventh cycle of test 4 and fourth cycle of test 6. At those points the samples entered states of unlimited flow, and flow failure ensued.

Cyclically loaded tests in which limited flow developed have been reported by Lee and Seed (Lee, 1970; Lee and Seed, 1967a; Seed and Lee, 1966, 1969). The granular sediments they used were subangular to subrounded fine uniform sands from the Sacramento River, Calif. The stress path of a typical test with stress reversals (test 114, see fig. 2 of Lee and Seed, 1967a) is plotted in figure 5. Owing to the frequency of loading (2 cycles per second) and the relatively slow speed of the recorder (2.5 cm/sec (1 in/sec)), stress paths can not be plotted as accurately for Lee and Seed's tests as they were for Castro's tests. Stresses at the conclusion of each loading segment were well defined, however, and are accurately plotted as the circled points in the figure. The test began with an isotropically consolidated sample ($D_r$=38 percent). During each cycle, pore pressure increased ($p$ decreased) in response to the tendency of the sample to compact. During the first eight and a half cycles, deformations were negligible. However, during the extensional loading segment of the ninth cycle, liquefaction occurred, and the sample entered a state of limited flow and rapidly strained 17 percent in axial extension before solidification occurred (point $s_1$) as a result of a dilatancy-caused drop in pore pressure. The inferred stress path during this phase of the test is shown by the inter-

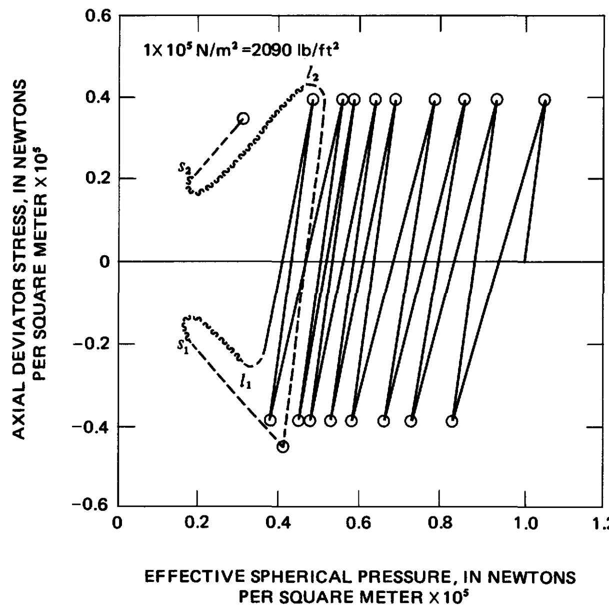


FIGURE 5.—Repeated episodes of liquefaction and limited flow generated by cyclic loading. Stress path from a cyclically loaded triaxial compression test with stress reversals. (Data from fig. 2, Lee and Seed, 1967a.)

mittent wavy line (fig. 5). Liquefaction recurred during the ensuing compressional loading (stress path through $l_2$ and $s_2$). The sample again entered a state of limited flow, strained 29 percent in axial compression, and then was restabilized by solidification. During each subsequent loading the sample reliquefied, rapidly deformed, and solidified at the opposite extremity of the developed strain excursion.

In another set of tests, Seed and Lee (1969) showed that deformations during cyclic loading are considerably greater for tests with stress reversals above some small threshold value than for tests without stress reversals or for tests with very small stress reversals. The behavior of a test with no stress reversal is shown by the stress path in figure 6 plotted from data (test 99) reported by Lee (1970, p. 321). During the first loading of the sample (stress path from 0 through $l$ to 1) liquefaction occurred; the sample entered a state of limited flow, strained 5.5 percent in axial compression, and then was restabilized by dilatancy-caused solidification. During unloading the pore pressure increased ($p$ decreased) as the dilatant tendency was relaxed; however, the stress path remained within the stable domain (path from 1 to 1$u$), thus preventing a recurrence of liquefaction. During the second loading (path from 1$u$ to

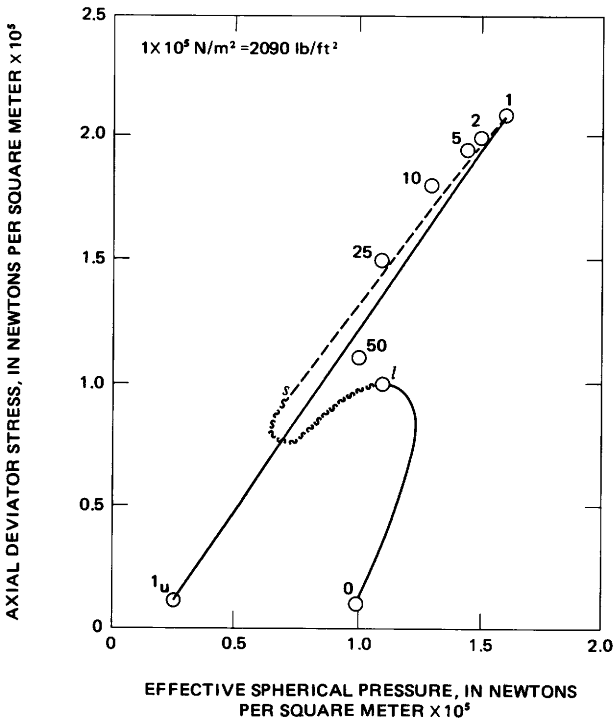


FIGURE 6.—Test data illustrating the necessity of stress reversals for cyclic loading to produce repeated cycles of liquefaction and limited mobility. Stress path from a cyclically loaded triaxial compression test without stress reversals. Liquefaction and limited flow occurred only during first loading. (Data from test 99, Lee, 1970.)

2), the applied shear stresses were resisted immediately by increased sample strength as a result of dilatancy-caused decreases in pore pressure. This sample hardening eliminated any possibility of liquefaction or flow during that segment of the test although some plastic straining did occur. During unloading the stresses again traced a path within the stable domain (from point 2 to about point $1u$). This behavior was repeated during each additional cycle. The stress conditions at maximum load for several of these cycles are shown by the correspondingly numbered points on the diagram. (The values decreased because the sample cross-sectional area increased during the test as a result of plastic straining.) In each cycle the stress paths returned approximately to point $1u$ during unloading. Thus, other than the single episode of liquefaction and flow deformation during initial loading, the sample did not reliquefy as did those in tests with appreciable stress reversal. These results clearly show that shear-stress reversals are necessary to produce repeated occurrence of liquefaction and limited flow.

The influence of a previous history of liquefaction on the susceptibility of a sand to reliquefaction at a later period has been demonstrated by Finn, Bransby, and Pickering (1970). The approximate stress paths for two liquefaction tests on the same Ottawa sand sample are plotted in figure 7 (data from fig. 5 (test 40) of Finn and others, 1970). The sample was formed in a triaxial compression device, saturated, and then consolidated under an isotropic stress of $2.0 \times 10^5$ $N/m^2$ (4,100 lb/ft$^2$) to a relative density of 50 percent. Cyclically reversing loads were applied to the sample. Pore pressure increased with each loading until liquefaction occurred during the extensional loading segment of the 25th cycle (light-line stress path in fig. 7). The sample flowed through an axial strain of 0.6 percent, at which time deformation was arrested by solidification. Several additional loading cycles were applied (stress paths not shown). With each load reversal, the sample reliquified, flowed, and then solidified at the opposite end of the strain excursion. After a total of 29 cycles the loading was stopped, leaving the sample stabilized at the compressional end of the strain excursion. The sample was then reconsolidated under an isotropic stress of $2.0 \times 10^5$ $N/m^2$ (4,100 lb/ft$^2$). This consolidation yielded a relative density of 60 percent. Cyclic loading was then resumed. This time liquefaction occurred during the extensional loading segment of the first cycle (heavy-line stress path in fig. 7). From this and similar tests, Finn, Bransby, and Pickering (1970) conclude that resistance to reliquefaction is substantially reduced by a previous episode of liquefaction.

## CONSEQUENCES OF LIQUEFACTION

Liquefaction by itself poses no particular hazard. In fact, during seismic shaking a liquefied layer at depth could act as an isolator, impeding the transmission of vibrational energy from underlying layers to structures founded at the surface (Seed, 1968; Ambraseys, 1973). Only when liquefaction leads to some form of permanent ground movement or ground failure does it become a serious problem. Three basic types of ground failures associated with liquefaction have been identified (Seed, 1968)—flow landslides, landslides with limited displacement, and quick-

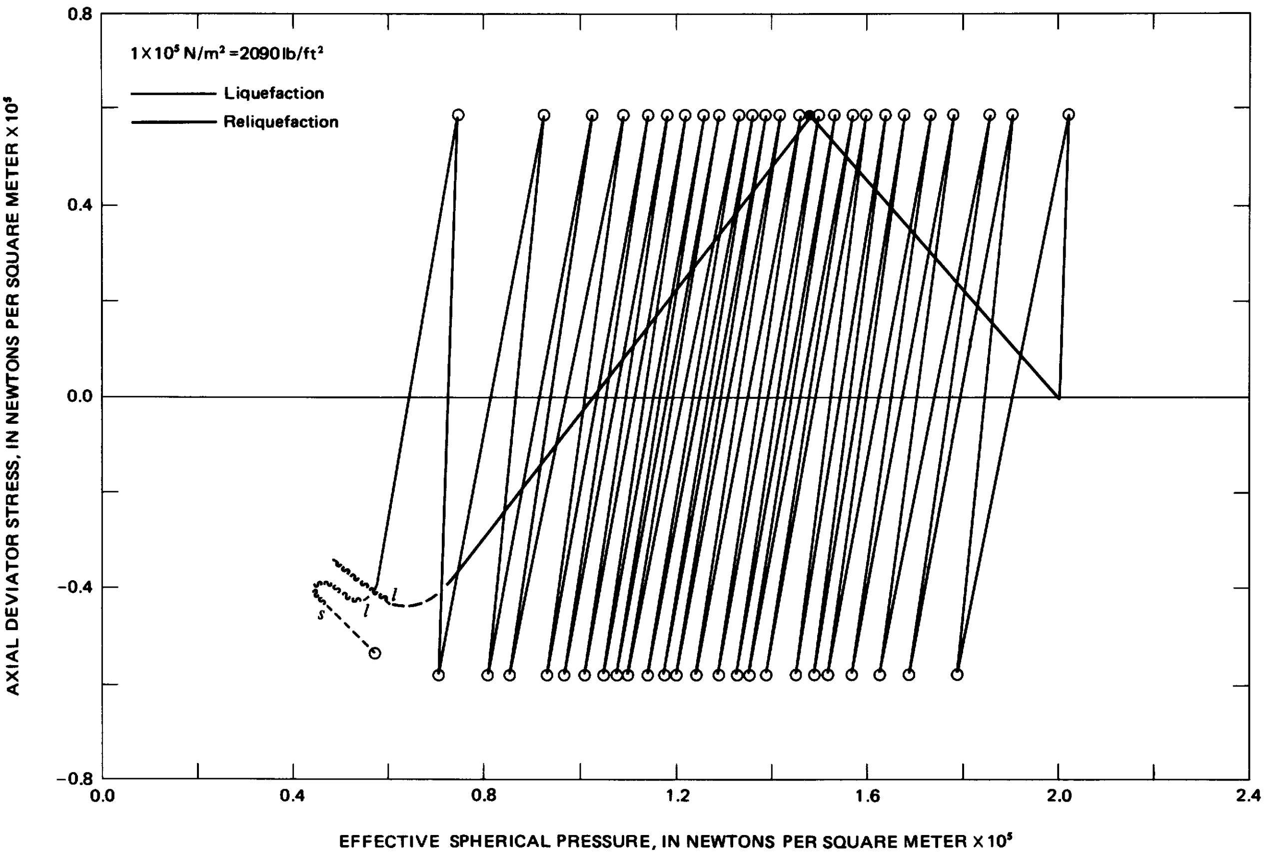


FIGURE 7.—Data showing that resistance to reliquefaction is reduced by a previous episode of liquefaction. Stress paths from cyclically loaded liquefaction and reliquefaction tests on the same Ottawa sand sample. Sample liquefied and flowed in limited flow in both tests. (Data from fig. 5, Finn and others, 1970.)

condition failures. In addition to these types of ground failure, ejection of water and sediments in the form of sand boils has been a source of damage associated with liquefaction during earthquakes (Ambraseys and Sarma, 1969).

## FLOW LANDSLIDES

Where conditions are favorable, liquefaction in the field may lead to a state of unlimited flow. In this situation, if the mobilized soil is unrestrained, sizeable masses of earth materials may travel long distances in the form of liquefied flows or blocks of intact materials riding on liquefied flows. In this type of failure, flow ceases only when the driving shear forces are reduced (such as by slope reduction) to values less than the viscous shear resistance of the flowing material.

The dimensions and character of typical flow landslides are illustrated in the few examples cited below.

Several flow landslides have occurred in the estuary section of the Dutch province of Zeeland (Koppejan and others, 1948), including some that have caused disastrous breaches in the dikes of that region. Dike slopes before failure were typically about 36 percent (20°), with banks as high as 40 m (130 ft). After failure, many slopes were 7 percent (4°) or less. Liquefaction of a loose granular layer underlying the slides areas was the apparent cause of the flow failures.

A flow failure occurred in the Fort Peck Dam, Montana, during construction of a hydraulic-fill embankment (Casagrande, 1965). A 250-m (1,700-ft)-long section of the upstream shell failed and moved 460 m (1,500 ft) upstream in about 4 min. Before failure, the slope of the upstream embankment was about 25 percent (14°); after failure slopes on the failed mass were generally less than 5 percent (3°). The cause of failure is believed to have been liquefaction of a sand zone in the shell and possibly a natural granular layer beneath the dam (Casagrande, 1965, p. 10).

Flow failures commonly are triggered by seismic shaking (Seed, 1968). Crandall (1908, p. 249) described one that occurred near San Francisco during the 1906 earthquake

> Northeast of Mount Olivet Cemetery there was an earth-flow in the sandy soil at the base of the San Bruno Mountains. The angle at which the materials slid was hardly more than 10 degrees [18 percent]. The sand and water forming this slide came out of a hole several hundred feet long and 150 feet [46 m] wide, flowed down the hill several hundred yards toward the cemetery, carried away a pile of lumber, and knocked the power-house from its foundations. The front of the mud-flow piled up in a bank when it reached the nearly level ground, and dammed up the mass behind it.

### LATERAL-SPREADING LANDSLIDES

Where conditions are favorable for liquefaction but the sediments are too dense to allow unlimited flow, limited flow may develop. On sloping ground this may allow finite downslope movements followed by dilatancy-caused solidification. Other factors that may also limit downslope displacement include rapid drainage and liquefaction of materials along only part of the potential failure plane (Seed, 1968). In many instances, however, dilatancy-caused solidification appears to be the most important factor in limiting displacement. One episode of limited flow may produce displacements of negligible importance; on the other hand, cumulative displacements generated by repeated episodes of limited flow, such as might occur during an earthquake or other source of repeated loading, could be substantial. Description of limited displacement landslides generated by liquefaction are common in the earthquake literature, but they are described under different nomenclature. For example, "earth lurches" (Richter, 1958, p. 124), land spreading (McCulloch and Bonilla, 1970), and lateral spreading (Oldham, 1899, p. 87–90; Youd, 1973) are, in fact, limited displacement ground failures associated with liquefaction. The term "lateral-spreading landslide" is used here to denote this type of failure because it is commonly used, it has been rigorously defined (Varnes, 1958, p. 29–32), and many failures described in the literature fit the description of lateral spreading given by Varnes.

Several important characteristics of lateral-spreading landslides associated with liquefaction can be deduced from the laboratory behavior described above. (1) Repeated episodes of limited flow can only develop if shear stress reversals occur during the loading sequence. These reversals are more easily accomplished beneath mild slopes where static shear stresses are small than beneath steeper ones. However, if the slope is too gentle, the gradient may not be sufficient to cause movement. Thus mild slopes should be most susceptible to this type of failure.

(2) With each occurrence of restabilization, the mobilized soil in the failure zone receives a dilatant impulse that could cause loosening if excess water were available to fill the created voids. Excess water is often available during earthquakes as a result of compaction of other granular materials in the section. Thus, it is possible for loosening to occur which, in turn, would allow displacements to increase with each episode of limited flow and could eventually lead to a condition of unlimited flow.

(3) At the conclusion of a series of limited-flow episodes, the soil in the failure zone is commonly left in a dilatant restabilized condition. The soil itself may be denser or looser or the same as it was before the disturbance, depending on whether pore water migrated into or out of the liquefied soil during shear. In any case, and even after reconsolidation, the soil may be left in a particularly vulnerable condition for reliquefaction during a subsequent disturbance.

(4) Repeated episodes of limited flow can continue only as long as strong ground shaking producing stress reversals continues. Thus, lateral-spreading landslides associated with liquefaction would normally move only during periods of strong shaking and restabilize immediately upon the cessation of shaking.

The following few examples of lateral-spreading ground failures illustrate their typical dimensions and character.

During the 1971 San Fernando earthquake, lateral-spreading ground failures occurred on both sides of Van Norman Lake (Youd, 1971; 1973; Smith and Fallgren, 1973; Proctor and others, 1972). Northeast of the lake the landslide was tongue shaped in plan, 1.2 km (4,000 ft) long, and about 0.3 km (1,000 ft) wide (average). The mean slope from head to toe of the failure was about 1.5 percent (0.9°). Horizontal displacements were as large as 1.9 m (5.7 ft), whereas maximum vertical displacements were only about 0.15 m (0.5 ft). The surface layer was fractured into several large blocks that slid downslope with very little tilting. Sand boils, indicative of increased pore pressures at depth, erupted at several points on and near the slide. The soil profile beneath the slide consisted of a firm surface layer overlying a soft saturated layer composed of sand and sandy silt. Liquefaction in the lower layer presumably led to the failure by repeated episodes of limited flow. Relict preearthquake fissures and sand boils that were found in postearthquake exploratory trenches are evidence that similar ground failures had occurred before at this site, most likely during previous earthquakes (Youd, 1972b).

West of the lake, the surface ruptures broke roughly parallel to the long free face formed by the west shore of Van Norman Lake and severely disrupted the newly constructed fill for the Jensen water-filtration plant. Maximum horizontal displacements were about 1 m (3 ft) near the free face and decreased with distance upslope. Sand boils were found near the toe of the landslide and on the fill in the southern part of the zone. A layer of saturated fine sand was discovered in postearthquake borings at depths of 2–3 m (6–9 ft) below the original ground surface (Proctor and others, 1972). The failure presumably was caused by liquefaction of this layer.

Reports from recent Alaskan earthquakes include numerous descriptions of lateral-spreading-type ground movements down mild slopes toward free faces; these ground movements were accompanied by fissuring and ejection of subsurface water. For example, McCulloch and Bonilla (1970, p. D–1) report that during the March 27, 1964 earthquake

> a general loss of strength [was] experienced by wet, water-laid unconsolidated granular sediments (silt to coarse gravel that allowed embankments to settle and enabled sediments to undergo flowlike displacement toward topographic depressions, even in flat-lying areas * * * Stream widths decreased, often about 20 inches [0.5 m] but at some places by as much as 6.5 ft [2.1 m], and sediments moved upward beneath stream channels * * * Ground cracks * * * commonly extended 500 ft [160 m], and occasionally 1,000 ft [320 m], back from streams * * * Sediment-laden ground water was discharged from the cracks.

During the 1906 San Francisco earthquake, several lateral-spreading ground failures occurred in the city of San Francisco. One, in an area of filled marshland, was approximately 2 blocks wide and extended over a 1.5-km (5,000-ft) distance from Eighth and Mission Streets to the vicinity of Fourth and Brannan Streets. Wood (1908) described this area as follows

> The fissuring and slumping and the buckling of block and asphalt pavements into little anticlines and synclines (arches and hollows), accompanied by small open cracks in the earth, characterize the land surface. This slumping movement or flow took place in the direction of the length of the area [down about a 0.8 percent (0.5°) slope], and its amount was greatest near the center or channel, where the street lines were shifted eastward out of their former straight courses by amounts varying from 3 ft to 6 ft [1 m to 2 m].

These examples, which are only a few of many that could be cited, show that lateral-spreading failures on mild slopes are very common during moderate and strong earthquakes.

### QUICK-CONDITION FAILURES

Seepage forces, caused by upward-percolating pore water, often reduce the strength of granular soils to a point of instability. This state is termed a "quick condition." Artesian pressures are one cause of upward seepage; seismic compaction of saturated granular materials at depth is another. This condition is generally restricted to sand layers of significant thickness that extend from below the water table to the surface.

Loss of bearing capacity is the most common type of failure produced by a quick condition. Buoyant rise of buried tanks and other vessels is another common effect. Shear deformations during such failures could be of either the unlimited- or limited-flow types. Bearing-capacity failures illustrative of this type of failure were reported during the 1964 Niigata, Japan, earthquake. In the city of Niigata, several high-rise apartment buildings rotated and subsided into the liquefied soil (Seed and Idriss, 1967). Railroad embankments, especially the Echigo line, subsided into the "liquefied ground" (Kobayashi, 1969).

## METHODS FOR EVALUATING LIQUEFACTION AND GROUND-FAILURE POTENTIAL

Existing methods for evaluating liquefaction and associated ground-failure potential are generally based on one of two criteria: (1) the number of cyclical loadings required to produce a liquefied condition or a certain amount of flow deformation considered to constitute failure; or (2) the smallest void ratio (critical void ratio) at which unrestrained liquefied flow (unlimited flow) can develop. Both of these criteria are controlled largely by the same general factors; however, the influence of these factors may be considerably different for each situation. For example, relative density is a primary factor controlling both liquefaction of sands under cyclical loading conditions and unlimited flow failure. Liquefaction can be induced by cyclical loading, however, at considerably greater relative densities than those at which unlimited flow can occur. Thus different criteria have been developed for each approach.

Seed and his coworkers (Lee and Seed, 1967a, b; Peacock and Seed, 1968; Seed and Idriss, 1967, 1971; Seed and Lee, 1966, 1969; Seed and Peacock, 1971) have developed the criteria for repeated loading. One application of this procedure has been the analysis of stability and deformation of earth dams during earthquakes (Seed, 1966; Seed and others, 1969, 1973). Similar procedures should be generally applicable for analyzing liquefaction potential and ground-failure movements beneath natural slopes.

In addition, Seed and Idriss (1971) formulated a "simplified procedure" for evaluating liquefaction potential. This method incorporates several simplifying approximations and is valid only for sediments with relative densities less than 80 percent that lie beneath level surfaces. Although the definition of liquefaction given by Seed and Idriss (1971, p. 1249, cited earlier) seems to indicate that the simplified procedure is generally valid for predicting ground-failure potential as well as liquefaction potential, examination of criteria used in formulating the procedure shows that this is not true. "Initial liquefaction" (Lee and Seed, 1967a, p. 49) was the criterion used by Seed and Idriss to determine the point at which liquefaction occurred in laboratory tests utilized in formulating the procedure. (Initial liquefaction is defined as the point at which a granular material exhibits any degree of liquefied flow during cyclic loading.) Although significant ground-failure movements may occur almost simultaneously with initial liquefaction in some loose granular soils (for example, relative densities less than 65 percent), this is not generally true for denser soils. For example, it may require many cycles of loading after liquefaction (or initial liquefaction) to produce significant ground-failure movements in denser sediments (for example, relative densities greater than 75 percent). In addition, ground slope, a factor of primary importance in evaluating ground-failure potential, is not incorporated in the simplified procedure. Thus, while the simplified procedure may be valid for evaluating liquefaction potential it is not necessarily valid for evaluating ground-failure potential. More rigorous methods, such as those developed for earth dams (Seed and others, 1973), are necessary for evaluating the latter potential.

The critical void ratio approach has been developed by Casagrande (1940) and Castro (1969) chiefly for monotonic loading conditions, the principal application being the differentiation of conditions under which unlimited and limited flow may occur. With additional research, particularly on the effect of repetitive loadings, this method could be made more general and could possibly be applied to evaluating type and amount of ground-failure movement after liquefaction. With such development the two methods would materially complement each other.

## SUMMARY

Explicit definitions for liquefaction, solidification, limited flow, and unlimited flow have been given and the relation between these phenomena and ground failure examined in an attempt to present a clear and integral description of liquefaction and its consequences. Also, methods for evaluating liquefaction and ground-failure potential have been briefly examined.

Liquefaction is defined as the transformation of a granular material from a solid state into a liquefied state as a consequence of increased pore-water pressures. Solidification is defined as the opposite process, that is, the transformation of a granular material from a liquefied state into a solid state. Once liquefied, a granular material is free to flow, provided there is sufficient gradient, until solidification occurs. If solidification occurs as a result of dilatancy with very little change in total stress, the condition is termed limited flow. If solidification occurs only when driving forces are reduced to a level less than the viscous shearing resistance of the material, the condition is termed unlimited flow.

Three types of ground failure conditions commonly follow liquefaction in the field. (1) Unlimited flow commonly leads to flow landslides. (2) Limited flow commonly leads to lateral-spreading landslides. One episode of limited flow may be inconsequential; however, a series of episodes, such as might occur during an earthquake, often leads to significant permanent ground movements. Many phenomena variously described in the earthquake literature as lurching or land spreading are, in fact, lateral-spreading landslides generated by liquefaction. (3) Quick-condition failures such as loss of bearing capacity form the third type of ground failures attributable to liquefaction.

Two approaches for evaluating liquefaction and ground-failure potential have been proposed. The procedure developed by Seed and his colleagues provides a method for evaluating liquefaction and ground-failure potential under cyclical loading conditions. The procedures developed by Casagrande and Castro were basically designed to differentiate conditions under which unlimited and limited flow occur during monotonic loading conditions. With further development these two approaches should become complementary and lead to improved understanding of the phenomena involved.

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