DESIGN PROBLEMS IN SOIL LIQUEFACTION

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ABSTRACT: An attempt is made to clarify some aspects of the problems encountered in evaluating the stability of embankments under conditions where a potential for soil liquefaction exists. It is suggested that at the present time, the most prudent method of minimizing the hazards associated with liquefaction-induced sliding and deformations is to plan new construction or devise remedial measures in such a way that either high pore water pressures cannot build up in the potentially liquefiable soil, and thus liquefaction cannot be triggered, or, alternatively, to confine the liquefiable soils by means of stable zones, so that no significant deformations can occur; by this means, the difficult problems associated with evaluating the consequences of liquefaction (sliding or deformations) are avoided. However, when large deformations can possibly be tolerated, it may be adequate and economically advantageous to simply ensure the stability of the embankment against major sliding after liquefaction has occurred. Evaluating this possibility requires a knowledge of the residual strength of the liquefied soil, and, while laboratory test procedures have been developed for determining such a strength, it is suggested that the establishment of a relationship between this property of a soil, as determined by field performance studies, and some in situ soil characteristic, such as penetration resistance, may provide the most practical method for evaluating residual strengths in cases where such values are required. Available data based on case studies is summarized and plotted in chart form for this purpose.

INTRODUCTION

In general, it may be said that there are two main problems confronting the soil engineer dealing with a situation where soil liquefaction may occur: (1) Determining the stress conditions required to trigger liquefaction; and (2) determining the consequences of liquefaction in terms of potential sliding and potential deformations. There is much evidence to show that if the pore pressures in a soil do not build up to high values, e.g., exceeding a pore pressure ratio of about 60%, liquefaction (in any of its forms) will not be triggered in the soil. If the soil does not liquefy in the sense that a high pore pressure ratio, r_u, is developed, then: (1) There is usually no problem of sliding since the soil retains high shear strength; and (2) there is usually no serious deformation problem. There are numerous examples of structures built of potentially liquefiable soils or constructed on potentially liquefiable soils that have stood for tens or hundreds of years without liquefaction occurring, simply because there has been no triggering mechanism strong enough to induce liquefaction. Thus, ensuring that liquefaction can not be triggered is a legitimate means of avoiding undesirable consequences. This can be achieved by designing on the principle of keeping the induced pore pressure ratio, r_u, well below 100%; achievement of this condition ensures that liquefaction will not occur, and thus it generally provides a stable and minimally deforming structure.

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Extensive work has been performed during the past 20 years to explore the conditions causing the development of liquefaction in clean sands and silty sands, and on the buildup of pore-water pressures leading to the onset of liquefaction, expressed as a condition where $r_u \simeq$ 100%. Thus, the ability of the profession to explore these conditions is relatively good. It is very good for level ground conditions because of the extensive data base of field case histories, and the principles involved in extending the method to embankments and sloping ground conditions are relatively well-established. The method has also been shown in several cases to provide results in good accord with some of the more important features of observed field performance of embankments.

An alternative design approach is to accept that liquefaction may be triggered in a potentially liquefiable soil, and to allow this condition to persist. Then the design problem becomes one of determining the potential for sliding and the potential deformations that may result from the inducement of liquefaction. In this case, it is necessary to be able to determine the strength and deformation characteristics of the liquefied soil. Significant differences exist within the profession at the present time about how these values should be determined (see following sections), and there are wide variations in professional opinions concerning the shear strength values appropriate for use in any given case. There is also fairly good agreement that for liquefied soils, "the prediction of deformations in soils not subject to flow failures is a very difficult and complex problem that is still far from being resolved" (NRC Committee on Earthquake Engineering 1985). Thus, once liquefaction occurs, the current ability of the geotechnical engineering profession to handle the problem of predicting the consequences deteriorates significantly.

The design of critical structures such as dams and nuclear power plants requires confident handling of both the stability and deformation problems, and given the present state of knowledge, it is the writer's view that the best way of ensuring that no undesirable consequences will develop is to design new embankments, or modify old embankments, in such a way that a condition of $r_u \simeq 100\%$ is never approached, except in limited and controlled zones of a structure. If we accept this point of view, then it is clear that the major emphasis in a soil liquefaction potential investigation should be placed on the triggering problem, and on exploring the conditions that cause sufficient pore pressure development to trigger liquefaction in a soil. It may be noted in passing that this does not imply that the inducement of a condition of $r_u \simeq 100\%$ is necessarily unacceptable. It is clear that the development of this condition in dense cohesionless soils is often of no practical significance, since the strains required to eliminate the condition are very small. Thus, dense cohesionless soils do not normally present problems in the design of dams or embankments, because they rarely, if ever, develop conditions where $r_u \simeq 100\%$, and, if they do, it will usually have no practical consequences.

The existence of different design goals with regard to the evaluation of liquefaction problems sometimes leads to conflicting requirements regarding the optimum conditions for achieving these goals. Thus, for example, there is an extensive body of laboratory test data to show that: (1) The higher the confining pressure, other things being equal, the more difficult it is to build up pore pressures in a soil as a result of cyclic shear stress applications; and (2) within the range of shear stress/normal stress ratios generally encountered in practice, the higher the initial shear stress in a soil element with a relative density above about 45%, the more difficult it is to build up pore pressures in the element by means of cyclic shear stress applications.

As a consequence, within a reasonable range of embankment heights and slopes often found in practice, larger embankments with somewhat steeper slopes, which create higher effective confining pressures and higher initial shear stresses, may make it more difficult (in terms of the required level of applied shear stresses) to build up pore pressures in sand deposits, and therefore more difficult to trigger liquefaction by inducing a condition of $r_u \simeq 100\%$, than would lower dams with flatter slopes. Since higher and steeper embankments may make it more difficult to build up pore pressure and trigger liquefaction, it would appear that these embankments can sometimes be constructed on sands with lower N_1 values and still not cause liquefaction to be triggered, than can embankments with lesser heights and flatter slopes. However, changing the height and slope of an embankment will also change the embankment response, thereby changing the level of applied shear stresses, and thus it is difficult to reach general conclusions concerning the relative effects of higher embankments and steeper slope conditions. There is no field evidence to indicate, however, that flattening the slopes or lowering the height of an embankment will reduce the possibility of triggering liquefaction, while there is limited evidence to suggest that the reverse may be true.

On the other hand, if liquefaction occurs and the liquefied soil develops a residual strength that is independent of confining pressure, then the larger the driving shear stresses in a soil structure, the more likely it is that either sliding or large deformations will develop. Thus, for high embankments and embankments with steeper slopes, higher residual strengths and thus higher N_1 values are required to prevent sliding than for smaller dams or embankments with flatter slopes.

Thus, if the problem of embankment stability on potentially liquefiable soils is approached from the point of view of evaluating what happens after the soil liquefies, it is concluded that steeper slopes and higher dams are more dangerous than flatter slopes and lower dams, or that a higher N_1 value is needed in the foundation soil for a high dam than for a low dam. This is a correct and logical conclusion, *if the soil in or below the dam is to be allowed to liquefy*. However, it seems to be highly questionable whether, at the current time, prudent design permits the development of a condition of $r_u \simeq 100\%$, except in certain limited zones, since it only leads to the creation of a situation, involving possible sliding and large deformation problems, which we have little confidence in our ability to handle.

Experience shows, for example, that reducing the driving stresses and ensuring a high factor of safety against liquefaction-type (flow) sliding does not necessarily prevent large deformations from developing if a soil liquefies. In fact, large deformations (5–10 ft) have occurred on slopes as flat as 2% (1 on 50), where the driving stress was as low as 60 psf and the post-earthquake factor of safety against sliding was probably

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greater than 2.0. Examples are the Juvenile Hall landslide in the San Fernando earthquake of 1971, and bridge foundation movements, such as those at the Snow River Bridge, in the Alaska earthquake of 1964. Furthermore, very low dams, with heights of 20 and 30 ft, are known to have failed and deformed excessively as a result of liquefaction, under relatively low levels of earthquake shaking (about 0.2–0.3 g). Thus, determining a residual strength, even if it is done reliably, is not necessarily a solution to the whole problem of embankment stability on potentially liquefiable soils; it is a potential solution in some cases (depending on the choice of residual strength values) to the flow slide evaluation problem, but it contributes little to the deformation evaluation problem. Thus it does not in itself produce an engineering solution to the practical problem of protecting public safety. Determining the residual strength of a liquefied soil and using it to evaluate slope stability is a potentially useful approach in cases where the prevention of major liquefaction-type slides is an acceptable solution to an embankment stability problem, but not to problems where large deformations and cracking may lead to failure. Thus, it may sometimes be applicable to flood-control or other dams with very large free-boards, or to tailings dams, where large deformations and cracking may be acceptable without permitting release of water or fluids from the reservoir. In these cases, the determination of a residual strength value for a liquefied soil can be the major aspect of a seismic stability evaluation.

In civil and geotechnical engineering, there are often different ways of approaching any given problem, and they often lead to similar results. However, the engineer's decision on methodology should be made in full awareness of all relevant facts, including the practicability of applying the methodology and the degree to which it is supported by case histories and past experience. Otherwise, it may be an interesting scientific exercise rather than the development of a good engineering solution (Peck 1978). Furthermore, it is important to adopt a design philosophy which effectively handles all recognizable aspects of a problem, and to be able to apply it with confidence that its results will last for a long time. This also means that its results must be supported by field performance data.

Recognizing this, it is important to document all available field performance for engineering structures and draw from it such lessons as will contribute to our knowledge of soil behavior. This means, from the standpoint of evaluating the residual (postliquefaction) strength of a soil, examining cases where major sliding has occurred due to liquefaction and where some conclusions can be drawn concerning the strength and deformation resistance of the liquefied soil. Unfortunately, such cases are rare. However, a small number of such cases do exist, for which the residual strengths of liquefied sands and silty sands can be determined with a reasonable degree of accuracy; SPT N_1 values are also available for these soils, permitting the development of an empirical relationship between the residual strength of liquefied sands, based on field case studies, and the N_1 values of the sands. It seems prudent to keep these values in mind when selecting residual strength values for other sand deposits in which liquefaction may be triggered, for whatever reason, whether it be sudden static stress applications or earthquake shaking.

TABLE 1.—Approximate Values of ΔN_1

Fines content (%) (1)	ΔN_1 (2)
10	1
25	2
50	4
75	5

In doing this it is also appropriate to recognize that, even for equal conditions of pore pressure generation-resistance or relative density, the penetration resistance of silty sands is lower than that for clean sands (Seed, et al. 1983; Skempton 1986). Thus the effective penetration resistance of a silty sand can be expressed for many practical purposes in terms of an equivalent clean sand value by use of the equation:

$$(N_1)_{\text{effective}} = (N_1)_{\text{measured}} + \Delta N_1 \quad (1)$$

where ΔN_1 depends on the fines content of the silty sand. Tentative values of ΔN_1 are given in Table 1, but judgment is required in the use of these values since fines may differ in their characteristics and effects from one soil to another.

In spite of this, an attempt to document case history data in this form is consistent with geotechnical engineering procedures for handling other design problems involving sands and silty sands, and this procedure is therefore followed in the following pages. In the interest of improved standardization, N_1 values are consistently related to those determined for an energy ratio of 60% in the SPT procedure and designated as $(N_1)_{60}$, as proposed by Seed, et al. (1985).

Case Studies of Liquefaction Slide Failures

Lower San Fernando Dam

Probably the best-defined case of a liquefaction-type slide is the failure of the upstream slope of the Lower San Fernando Dam just after the San Fernando (California) earthquake of 1971 (Seed, et al. 1975; Seed 1979). A representative cross section of the embankment of the dam and the approximate position of the surface of sliding are shown in Fig. 1. Field studies performed after the failure showed that liquefaction in this case extended over the greater part of the base of the upstream shell, with a short nonliquefied zone about 50 to 80 ft long near the toe. Thus the situation after the earthquake triggered the development of a zone of liquefaction within the embankment was essentially as shown in Fig. 1. Since sliding occurred relatively slowly, about one minute after the end of the earthquake shaking, the static forces tending to cause sliding were apparently just equal to the combination of the strength mobilized in the nonliquefied soil near the toe and the crest and the residual strength of the liquefied sand. From the known strengths of the nonliquefied zones it is a simple matter to calculate that, in this case, the residual strength of the liquefied sand at the start of sliding was about 750 psf. It may have been reduced as sliding progressed.

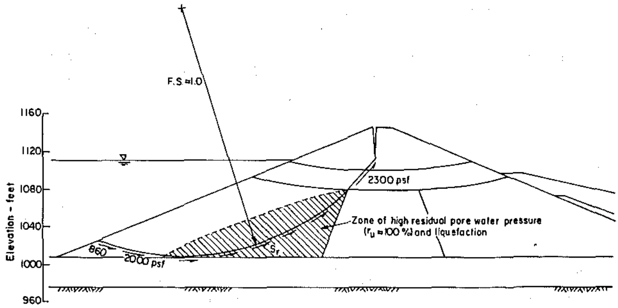

FIG. 1.—Cross Section of Lower San Fernando Dam at End of Earthquake

Numerous borings made in the downstream shell of the embankment following the earthquake, in material similar to that in the upstream shell, show that the average value of $(N_1)_{60}$ for the sand comprising shells is about 16, and field tests indicated that the relative density of the sand was about 50 to 55%. Both the relative density and the penetration resistance may have been slightly lower before the earthquake, with values of about $D_r \simeq 50\%$ and $(N_1)_{60} = 15$, respectively. The $(N_1)_{60}$ value of about 15 is also indicated by SPT tests performed before the earthquake.

Sheffield Dam

The Sheffield Dam failed near the end of an earthquake near Santa Barbara, California in 1925, as a result of a slide of the entire embankment on a liquefied layer covering essentially the entire base; in effect, the embankment was pushed downstream by the water pressure acting on the upstream face (Seed, et al. 1969). The conditions at the time of failure are shown in Fig. 2. A simple calculation shows that if liquefaction occurred all along the base, the residual strength of the liquefied soil when sliding occurred would be about 50 psf.

A study performed by the U.S. Army Corps of Engineers (1949) concluded that sliding occurred on a liquefied layer of silty sand having a relative density of about 40%. This would correspond to a value of $(N_1)_{60}$ for a clean sand of about 6 to 8.

Fort Peck Dam Slide

A major slide occurred in the upstream shell of the Fort Peck Dam, near the end of construction of this hydraulic fill structure in 1938 (U.S.

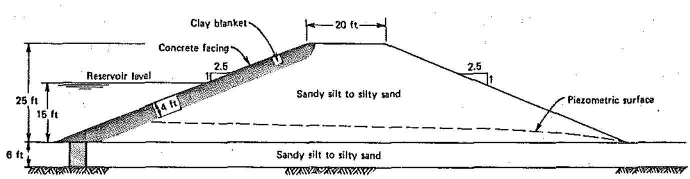

FIG. 2.—Cross Section through Sheffield Dam at Time of Failure

Army Corps of Engineers 1939; Casagrande 1965). From the configuration of the slide material after failure, Bryant, et al. (1983) concluded that the residual strength of the liquefied sand was about 240 psf. Other studies indicate a pre-sliding driving stress of about 700 psf; a reasonably conservative value is probably about 600 psf.

It is believed that, in this case, the slide occurred due to liquefaction of sand in the foundation. Studies made by the U.S. Army Corps of Engineers, both soon after the slide occurred and during a re-evaluation of the stability of the dam in 1976 (Marcuson and Krinitzsky 1976), indicate that the relative density of the sand was probably about 45 to 50%. This would correspond to a value of $(N_1)_{60}$ for a clean sand of about 12.

Mochi-Koshi Tailings Dam Slide

A slide occurred due to liquefaction of the soil in a tailings dam in Japan in the near Izu-Oshima earthquake of 1979 (Marcuson, et al. 1979; Ishihara 1984). Both Lucia (1981) and Bryant, et al. (1983) concluded that the residual strength of the liquefied tailings was about 210 psf. Penetration tests on the tailings indicate a penetration resistance $(N_1)_{60}$ of about 1, but allowing for the fact that the tailings consisted of very fine-grained (silt-size) particles, the equivalent $(N_1)_{60}$ value is probably about 6.

Juvenile Hall Landslide, San Fernando

An extremely interesting landslide, involving liquefaction, but not resulting in a flow-slide type of failure, is the Juvenile Hall slide which occurred in the San Fernando earthquake of 1971 (Youd 1971). A mass of soil about 20 ft thick and about 3,000 ft long moved laterally about 5 ft on a gentle slope of about 1.5°. The soil at the base of the slide mass was a saturated sandy silt with a SPT $(N_1)_{60}$ value of about 2. This would correspond to an equivalent sand value of $(N_1)_{60} \simeq 6$.

It is readily apparent from the very gentle slope that the shear stress on the base of the slide mass was only about 55 psf, and, even over a length of 800 ft, this would not be sufficient to overcome passive pressure acting on the end of the slide mass and the residual strength of the soil along the base. Apparently, sliding could only occur when the inertia forces induced by the earthquake motions were operative in the down-slope direction.

The maximum ground surface acceleration induced by the earthquake was probably about 0.6 g. Analyzing this situation using a Newmark-type deformation analysis leads to the conclusion that the residual strength of the sandy silt must have been about 140 psf for surface displacements of about 5 ft to have occurred.

Snow River Bridge Slide Movements

Lateral deformations similar to those which occurred in the Juvenile Hall landslide also occurred at the site of the Snow River Bridge in Alaska during the earthquake of March 19, 1964 (Ross, et al. 1969). In this case the river bed moved downstream about 10 ft, carrying the piers for the new bridge with it. The soil involved in the lateral slide movement was a gravelly sand with a penetration resistance $(N_1)_{60} \simeq 7$, and the slope of the ground is estimated to be not greater than about 1–1-1/2°. The residual strength of the liquefied sand was very low, probably about 50 psf, for movements of about 10 ft to have occurred.

Calaveras Dam Site

A liquefaction-type slide occurred in the upstream shell of the Calaveras Dam as it approached a height of 200 ft in 1918 (Hazen 1918). The dam was a hydraulic fill structure, and it was subsequently reconstructed using rolled fill construction. From the configuration of the slide mass, the residual strength of the liquefied sand is estimated to be about 750 psf, and tests performed in recent years show that the SPT $(N_1)_{60}$ value for the hydraulic sand fill in the original structure was probably about 12.

Dike Failure along Solfatara Canal

A dike failure occurred due to liquefaction along the bank of the Solfatara Canal in Southern California in the El Centro earthquake of 1940 (Ross 1968). The dike was about 7 ft high and the average shear stress at the base of the dike was about 130 psf; however, the residual strength was significantly less than this. The relative density of the sand foundation was measured to be about 30%; this would correspond to an $(N_1)_{60}$ value of about 5.

Slope Failure along Bank of Lake Merced, San Francisco

Major flow slides occurred in a sand deposit along the bank of Lake Merced, California in the San Francisco earthquake of 1957 (Ross 1968). Since the duration of shaking was only about 4 seconds, it seems clear that most of the slide movements (about 100 ft) must have occurred after the earthquake motions had stopped. The residual strength of the slide mass has been estimated to be about 100 psf and the penetration resistance of the sand was found to be about $(N_1)_{60} \simeq 5$.

Uetsu Railway Embankment

A sand fill placed to serve as a 33-ft high railway embankment failed during the 1964 Niigata earthquake in Japan (Yamada 1966). The embankment was constructed across a rice field and the bottom portion of the embankment was saturated. The liquefied sand flowed about 400 ft over ground which sloped at about 2°, and came to rest at a slope angle of about 4°. Lucia (1982) estimated that the residual strength of the liquefied sand was about 35 psf. The $(N_1)_{60}$ value for the sand is unknown. However, since the embankment had performed satisfactorily under train loadings before the earthquake, it is unlikely that the $(N_1)_{60}$-value for the sand was less than about 3.

Kona Numa Railway Embankment

Another small railway embankment, 10 ft high, at Koda Numa, Japan failed during the 1968 Tokachi-Oki earthquake (Mushima and Kimura 1970). The soil was a fine to medium sand which liquefied during the earthquake. The embankment failed by flowing in both directions, from the center line, over level ground. The liquefied material flowed about 60 ft, coming to rest at a slope of about 4°. Lucia (1982) estimated that the residual strength of the liquefied sand was about 25 psf. No data is

TABLE 2.—Residual Strengths of Liquefied Sands

Structure (1)	Relative density (%) (2)	Equivalent clean sand, $(N_1)_{60}$ (3)	Residual strength (psf) (4)	Cause of sliding (5)
Lower San Fernando	≈50	≈15	≈750	Earthquake
Sheffield	≈50	≈6	≈50	Earthquake
Fort Peck Dam	≈45	≈11	≈600	Construction
Mochi-koshi Tailings Dam	—	≈6	≈250	Earthquake
Juvenile Hall Slide	—	≈6	≈140	Earthquake
Snow River Bridge	—	≈5	≈50	Earthquake
Calaveras Dam	—	≈12	≈750	Construction
Dike, Solfatara Canal	≈30	≈5	≈130	Earthquake
River Bank, Lake Merced	≈40	≈5	≈100	Earthquake
Uetsu Railway Embankment	—	≈3	≈35	Earthquake
Koda Numa Railway Embankment	—	≈3	≈50	Earthquake
Kawagishi-cho	—	≈4	≈120	Earthquake

available concerning the penetration resistance of the sand, but, again it is not likely to be less than about 3 or 4 in a railway embankment of this type.

Building Foundation Failure at Kawagichi-cho

During the Niigata earthquake, a 4-story apartment building suffered a foundation failure and overturned as a result of liquefaction of the supporting foundation sand, for which $(N_1)_{60}$ is not likely to be less than 4. The average base pressure can be estimated to be about 600 psf, inducing an average shear stress in the foundation sand of about 120 psf.

Summary of Liquefaction Slide Data on Residual Strengths

The results of the evaluations of residual shear strength for the liquefied soils described above and the equivalent clean sand $(N_1)_{60}$ values of the soils are summarized in Table 2. The relationship between the residual strengths of the liquefied sands and the equivalent clean sand $(N_1)_{60}$ values for the soils involved is shown in Fig. 3. There is considerable scatter in the results, possibly reflecting differing degrees of water content redistribution resulting from different degrees of soil stratification, and, to some extent, whether the values were determined from conditions at the beginning of sliding or from conditions at the end of sliding. Nevertheless, they reflect field performance for a number of sands and silty sands, and thus provide a useful guide for engineering decisions concerning the residual strengths which may be developd in liquefied sands and silty sands for other deposits.

Clearly, there is a considerable degree of judgment involved in interpreting some of these case histories, and the range indicated in Fig. 3 may need to be broadened in the light of other interpretations or additional data. However, it is believed to represent a reasonable guide for use at the present time.

Residual Strength of Liquefied Soil Determined by Laboratory Tests

It has recently been proposed that the shearing resistance of liquefied soil can alternatively be determined directly from the results of consol-

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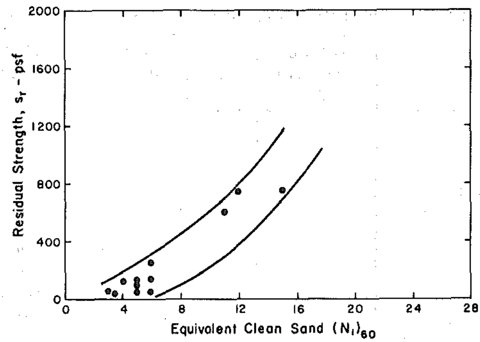

FIG. 3.—Tentative Relationship between Residual Strength and SPT *N*-Values for Sands

idated-undrained laboratory triaxial compression tests on undisturbed samples by determining the steady-state strength at which the soil will deform continuously without change in this resistance to deformation (Poulos, et al. 1985). Determination of this strength requires that appropriate corrections be made to the results of laboratory tests to allow for densification of the test specimens during sampling, during handling, and during reconsolidation in the laboratory to the stress conditions existing in the field. In the proposed procedure, the steady-state strength of a good quality undisturbed sample is determined at the laboratory void ratio after reconsolidation in the laboratory. It is then assumed: (1) that there is a unique relationship (the steady-state line) between steady-state strength and void ratio; (2) that the slope of the steady state line is the same for reconstituted samples of the sand as it is for undisturbed samples of that sand; and (3) that the slope of the steady-state line is independent of the method by which samples are reconstituted in the laboratory. Thus, by performing tests on reconstituted samples, the slope of the steady-state line for these samples can be established and used to predict the steady-state strength of the undisturbed sample at the void ratio corresponding to its in situ condition. The procedure for accomplishing this is shown in Fig. 4. It would certainly be advantageous to be able to determine the post-liquefaction resistance of soils in this way; however, available experience (Von Thun 1986) seems to indicate that in many cases the procedure leads to significantly higher values of residual strength than those indicated in Fig. 3.

This may be due to the fact that the slope of the steady-state line may not be independent of the method of sample preparation (N. Dennis, personal communication, 1986) or because one of the key assumptions in the presently-proposed use of this procedure is the concept that the void ratio of a sand deposit, after it liquefies, is the same as that of the soil before it liquefied; and it is not clear that this is necessarily the case. Even under constant volume (undrained) conditions, it is possible that there is a redistribution of water content in sand samples in the laboratory (Casagrande 1978; Castro 1975; Gilbert 1984) and in sand layers in the field. In fact, shaking table tests on stratified sand layers (Liu and

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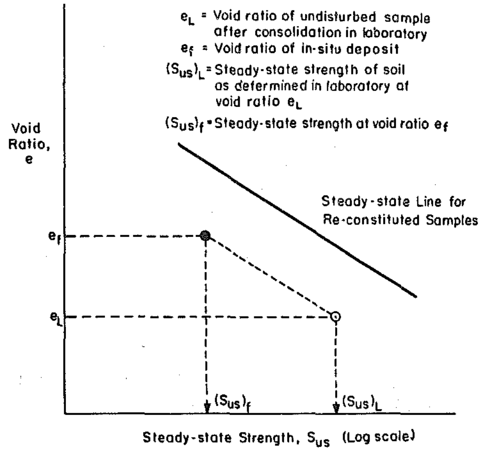

FIG. 4.—Procedure for Determining Steady-State Strength for Soil at Field Void Ratio Condition (after Poulos, et al. 1985)

Qiao 1984; see Fig. 5) show clearly that even under undrained conditions, in stratified sands water may accumulate below an impervious zone and form a water interlayer, as a result of water content redistribution. The procedure by which this may occur has been described in a report by the NRC Committee on Earthquake Engineering (1985), and by Whitman (1985), see Fig. 6; it involves the densification of sand in the lower part of a layer and the corresponding loosening of the sand in the upper part of the layer. In the extreme, the sand at the top of the layer may consist only of void space so that its void ratio becomes infinitely large and a thin zone consists only of water. This apparently is the condition described by Liu and Qiao.

Recognizing that this may also occur in the field under earthquake loading conditions, it becomes apparent that the lowest strength of the liquefied soil will be that for the loosened zone of sand at the top of a

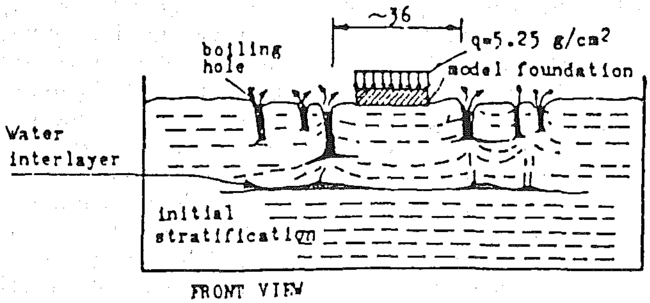

FIG. 5.—Results of Shaking Table Test on Deposit of Stratified Sand (after Liu and Qiao, 1984)

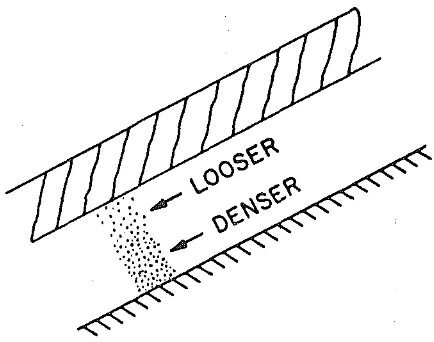

FIG. 6.—Example of Potential Situation for Mechanism B Failure Arising from Rearrangement of Soil into Looser and Denser Zones

layer, where the void ratio near the end of earthquake shaking may be higher than the initial (pre-earthquake) void ratio of the sand. Even if the validity of steady-state theory is accepted, therefore, (and the writer believes it to provide a very reasonable basis for understanding the strength of liquefied sands), it is not necessarily appropriate to correct steady-state strengths to the pre-earthquake void ratio of a sand deposit. In fact, if the lowest strength which controls stability is to be determined, the strengths determined by laboratory tests in which no water content redistribution occurs should be corrected to a void ratio corresponding to that of the loosest sand zone that may exist in the field near the top of a layer and below a more impervious boundary; this void ratio may apparently approach infinity in some cases (see Fig. 5), and its value is likely to depend on the nature and degree of stratification of the field deposit and its relative density, among other factors. There seems to be no good basis for anticipating the extent of such water content redistribution at the present time, other than evaluating its effects from the performance of field deposits in which flow slides due to liquefaction are known to have occurred.

This simply means that for cases where water content redistribution may occur, the steady-state strength of a soil at its pre-earthquake void ratio may be viewed as an upper bound value, and that the actual strength which the liquefied sand will mobilize may be significantly lower than this value, depending on the extent to which water content redistribution occurs in the field. Viewed in this light, there may be many steady-state strengths, depending on the void ratio that an engineer considers to represent the conditions in the critical zone of a deposit after liquefaction has occurred. For this reason it seems preferable to refer to the post-liquefaction strength of a sand as the residual strength of the soil. This may certainly be considered as a special value of the steady-state strength, but it corresponds to the steady-state strength at some unknown void ratio which is higher than the pre-earthquake void ratio, and may apparently in some cases be as great as infinity.

Under these conditions, even with the acceptance of the assumptions involved in steady-state theory, there seems to be no recourse for the practicing engineer interested in the field behavior of sand deposits than to accept the concept that the effects of water content redistribution, to whatever extent it occurs in nature, can only be evaluated at the present time by back-analyses of previous flow slides as described in the previous section. This is not a limitation of steady-state theory, but rather of our current inability to predict water content redistribution in soil deposits subjected to earthquake shaking under undrained conditions. This problem may not exist in cases where liquefaction is induced by static loading, and the steady-state strengths determined by appropriate laboratory tests should be applicable to problems of this type.

Deformations of Embankments Overlying Liquefied Soil

From time to time a problem will arise in which it may be necessary to determine the seismic stability of an embankment overlying a liquefied sand layer in the foundation. The sand layer may be so loose that it liquefies early in the earthquake and its strength then drops to a residual value, as indicated by the data in Fig. 3.

With a small embankment, as indicated in Fig. 7, and a sand layer located well below the surface, it may well be possible to show that, even if the liquefied sand has no significant residual strength, in the absence of any inertia forces the embankment still has an ample margin of safety against a liquefaction-type slide, due to the fact that the passive pressure at the toe of the slide far exceeds the active driving pressure at the head of the slide. It may also be argued that, because of the damping effect of the liquefied sand layer, no significant inertia forces should be induced in the soil overlying the liquefied layer, and thus no significant deformation of the slide mass is likely to occur.

Note that this rationale is not supported by the observed field performance of slide masses at the Juvenile Hall landslide in San Fernando (Youd 1971) or by the movements of the upper layers of soil at the site of the new Snow River Bridge in Alaska (Ross, et al. 1969). In these cases, the ground surface moved between 5 and 10 ft even though there was virtually no driving stress developed on the base of the slide block and the slope of the ground surface was very flat (1–2°). This is a form of lateral spreading, and it seems to require consideration of large inertia forces acting on long slide masses, as well as low residual strengths, to explain the magnitude of the observed deformations.

Thus, special caution is required in analyzing the stability of embankments under these conditions, especially in cases where large deformations constitute an unacceptable type of performance. It should also be noted that in cases where lateral spreading occurs due to earthquake

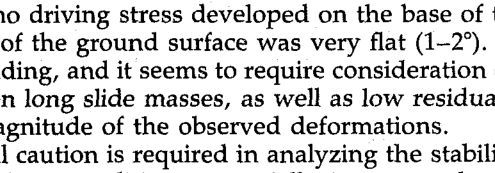

FIG. 7.—Schematic View of Low Embankment Underlain by Very Loose Sand Layer

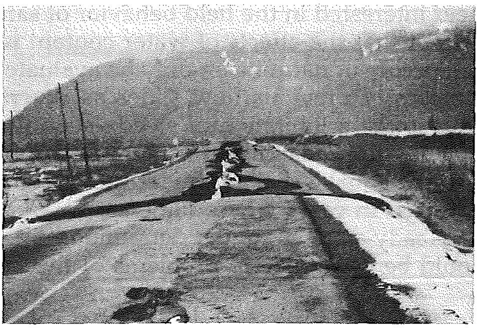

FIG. 8.—Cracking of Embankment Associated with Lateral Spreading of Embankment in Alaska Earthquake (1984)

shaking, the movements are often accompanied by transverse cracking of the embankment, as shown in Fig. 8. This type of deformation behavior is especially undesirable in small embankment dams, since it could readily lead to release of water through the transverse cracks, and thus to erosion and failure.

Special care is apparently necessary in evaluating the potential for deformations under conditions of this type.

Desirable Conservatism in Liquefaction Analysis of Embankment Stability

In the preceding pages it has been postulated that with the current state of knowledge, the best way to avoid undesirable and detrimental deformations of earth structures due to soil liquefaction is to prevent the triggering of liquefaction in the first place. It is also suggested that the current ability of the geotechnical engineering profession to predict the deformations of earth structures following liquefaction is quite poor and not sufficiently well-developed or proven to provide results with sufficient reliability for design or safety evaluation purposes in dealing with critical structures.

The problem of predicting deformations following liquefaction can be broken into two categories, however:

1. Deformations which occur due to liquefaction of a substantial body of soil partway through a period of strong earthquake shaking, so that movements occur due to the effects of both static and inertia forces acting on a composite system of liquefied and non-liquefied soils. This is, indeed, a formidable problem for which reliable deformation-evaluation techniques are poorly developed.
2. Deformations which occur in cases where liquefaction may occur in a substantial body of soil near the conclusion of strong earthquake shaking, so that subsequent deformations are virtually unaffected by the remaining very small inertia forces which follow the onset of liquefaction, and are due entirely, for practical purposes, to the effects of static

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stresses acting on the composite mass of liquefied and nonliquefied soil. This is a much simpler problem and the estimation of potential deformations in such situations is probably within the current capability of geotechnical engineering practice. Consideration would have to be given to the possible effects of water content redistribution, which may lead to failures or large deformations long after the earthquake motions have ceased (up to 24 hours later, as evidenced by the post-earthquake failure of the Mochi-Koshi tailings dam in Japan in 1979), to evaluation of an appropriate residual strength for the liquefied soil before any drainage occurs and to its possible changes with time, and to the stress-deformation relationships of the liquefied and nonliquefied soils. In the light of these considerations, the overall stability of the soils involved could be evaluated by accepted methods of stability analysis, and, if major sliding is not likely to occur, conservative estimations of deformations could be made. This might be accomplished, for example, by examining the stability of the structure for several assumptions concerning the resistance provided by the liquefied soil zone:

a. Assuming that the full residual strength of the liquefied soil is mobilized to prevent sliding. If the computed factor of safety is less than or close to 1.0 under these conditions, then sliding and large deformations must be anticipated.
b. Assuming that the resistance to deformation of the soil in the liquefied zone is zero. If, under these conditions, the computed factor of safety is significantly larger than 1.0, then the deformations are controlled by the strength and deformations in the nonliquefied soil, and the deformations are likely to be small.
c. If the results of the above analyses show that the slope is only stable if the liquefied soil makes some contribution to the resistance to sliding, then the amount of the sliding resistance which must be mobilized in the liquefied zone to produce a stable condition can be computed, and the shear strain which would have to develop in the liquefied soil in order to mobilize this resistance could be estimated conservatively. From a knowledge of this strain, the potential deformation of the slope or embankment could then be evaluated.

The evaluation of potential deformations in this way does not appear to be beyond the scope of available geotechnical abilities and could well be applied in cases where the primary condition for its applicability is satisfied: that is, when liquefaction occurs just at the end of earthquake shaking. This condition is achieved when the computed factor of safety against liquefaction (defined as a condition where the pore pressure ratio $r_u \simeq 100\%$) is close to unity; in these terms a factor of safety less than unity indicates that liquefaction, in the form of $r_u \simeq 100\%$, is achieved partway through the period of earthquake shaking.

Thus, when the computed factor of safety against the occurrence of a condition of $r_u \simeq 100\%$ is close to unity, the determination of the resulting deformations may reasonably be considered to provide an adequate evaluation of embankment or slope stability. In designing new structures, it would normally seem prudent to plan the design to prevent this condition from occurring. However, in dealing with existing structures which are marginally safe against the triggering of liquefaction, it may well provide an adequate basis for seismic stability evaluation, provided, of course, that the estimated deformations are acceptably small.

Conclusions

In the preceding pages an attempt has been made to clarify some aspects of the problems encountered in evaluating the stability of embankments under conditions where a potential for soil liquefaction exists. It is suggested that, at the present time, the most prudent method of minimizing the hazards associated with liquefaction-induced sliding and deformations is to plan new construction or devise remedial measures in such a way that either high pore water pressures cannot build up in the potentially liquefiable soil, and thus liquefaction cannot be triggered, or, alternatively, to confine the liquefiable soils by means of stable zones so that no significant deformations can occur; by this means, the difficult problems associated with evaluating the consequences of liquefaction (sliding or deformations) are avoided.

When large deformations can possibly be tolerated, however, it may be adequate and economically advantageous to simply ensure the stability of the embankment against major sliding after liquefaction has occurred; evaluating this possibility requires a knowledge of the residual strength of the liquefied soil, and while laboratory test procedures have been developed for determining such a strength, it is suggested that the establishment of a relationship between this property of a soil, as determined by field performance studies, and some in situ soil characteristic such as penetration resistance may provide the most practical method for evaluating residual strengths in cases where such values are required. Available data based on case studies is summarized and plotted in chart form for this purpose.

It is hoped that the discussion and results presented will help to clarify some of the conceptual differences which currently exist among geotechnical engineers with regard to the subject of soil liquefaction and its effects. It would appear that a principal basis for differing points of view rests on the degree to which laboratory tests are considered to be representative of field conditions, a subject discussed by many soil engineers over a long period of time, ranging from Terzaghi (1936) to (more recently) Peck (1978). Laboratory tests play a major role in geotechnical engineering studies of all types, but they only provide reliable data if they reproduce faithfully all essential aspects of the field situation they are intended to represent. Where doubt exists on this matter, case studies have necessarily provided the key to understanding field behavior.

In the view of the writer, the present situation can perhaps best be summarized as follows:

1. We need field performance data to tell us how soils really behave in the field.
2. We need in situ and laboratory tests, together with analyses, to provide us with an understanding of why soils behave the way they do in the field and thus to enable us to extrapolate available field experience

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to new situations for which no experience exists.

In the best of all worlds, we would be able to obtain the same values of the soil properties which control field behavior, either by field performance studies or by in situ or laboratory tests, and we would be able to understand, predict and explain field behavior completely on the basis of in situ or laboratory test data. This should be the goal we strive for, and we should work continually to improve our field and laboratory test procedures until we reasonably well achieve it. Unfortunately, to compound our problems, we do not work in an ideal world. We deal with complex deposits which are usually nonuniform and often highly stratified and sensitive to disturbance. Thus, we need in situ tests to identify the stratigraphy and nonuniformity, to determine parameters which may be changed by sampling, to provide convenient indices of soil behavior, and to ensure that our field case studies and laboratory studies address the correct problem soils.

All procedures have an important role in geotechnical engineering, and good engineering requires the optimum use of all the tools at our disposal. However, in developing solutions to practical problems involving the possibility of soil liquefaction, it is the writer's judgment that field case studies and in situ tests provide the most useful and practical tools at the present time. I recognize that this may not always be the case, however, and I look forward to the day when all the pieces will fit together to provide a coherent body of information which will provide both a reliable basis for predicting the field performance of soils and earth structures, and a good understanding of the reasons why this should be so. Until all uncertainties are resolved, however, I believe that it is prudent to pay careful attention to past field performance data, coupled with meaningful in situ testing, in developing solutions to soil liquefaction problems in engineering practice.

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