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DESIGN PROBLEMS IN SOIL LIQUEFACTION

by

H. BOLTON SEED

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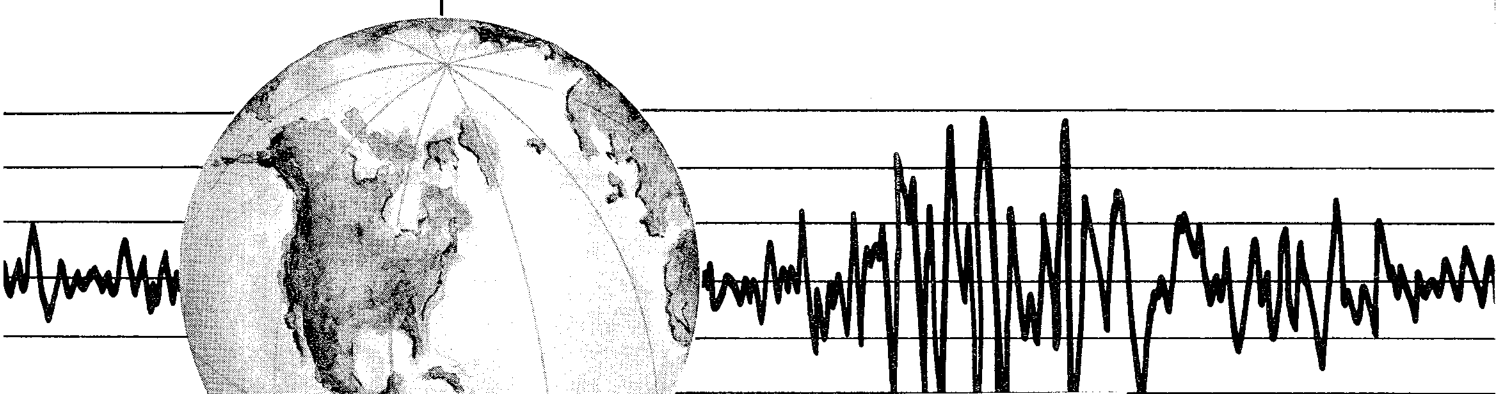

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The most prudent method for evaluating t he stability of embankments under conditions where a potential for soil liquifaction exists is to prevent buildup of high pore water pressures and prevent triggering of liquifaction. Where large deformations could possibly be tolerated, it may be adequate to ensure stability against sliding after liquifaction occurs. Evaluating this possibility requires knowledge of the residual strength of the liquified soil. Available data from field observations of the residual strength of liquified soils is summarized and plotted in chart form relating residual strength to some in-situ soil characteristic such as soil penetration resistance.

The general principles of a design philosophy for handling liquifaction problems at the present state of knowledge is presented; however the ability of the profession to predict deformations of soil structures after liquifaction suggest that this is not a suitable method for critical structures, but may well provide an economically feasible solution of design problems where the risk of large deformations are considered acceptable.

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Design Problems in Soil Liquefaction

by

H. Bolton Seed[1]

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, say exceeding a pore pressure ratio of about 60%, liquefaction 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 no serious deformation problem.

There are numerous examples of structures built of liquefiable soils or constructed on liquefiable soils that have stood for tens or hundreds of years without liquefaction occurring, simply because there has been no triggering mechanism sufficiently strong 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

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100%; achievement of this condition ensures that liquefaction will not occur and thus it generally provides a stable and minimally deforming structure.

There has been extensive work performed during the past 20 years to explore the conditions causing the development of liquefaction in sands and silty sands and on the build-up 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 to provide results in good accord with some of the more important features of observed field performance of embankments in a number of cases.

An alternative design approach is to accept that liquefaction may be triggered in a potentially liquefiable soil and 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 general 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 author'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 and limited field experience 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;

and 2. 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.

Thus it follows that large embankments with steeper slopes, which create higher effective confining pressures and higher initial shear stresses, make it more difficult to build up pore pressures in most sand deposits, and therefore more difficult to trigger liquefaction by inducing a condition of r_u = 100% than do low dams with flatter slopes. Since high and steeper embankments make it more difficult to build up pore pressures and thus trigger liquefaction, it follows that these embankments can be constructed on sands with lower values of the normalized penetration resistance, N_1, and still not cause liquefaction to be triggered, than can embankments with lesser heights and flatter slopes.

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, both of which are conducive to the development of large driving stresses, 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 <u>after</u> the soil liquefies, it is concluded that steep slopes and high dams are more dangerous than flat slopes and low 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 present 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 to 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 failure of safety against sliding was probably greater than 2.5. 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 0.2g to 0.3g). 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 freeboards or to tailings dams, where large deformations and cracking may be acceptable without permitting release of water or fluid 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 handles effectively all recognizable aspects of a problem and 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 (post-liquefaction) 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 a 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.

In doing this it is also appropriate to recognize that even for equal conditions of liquefaction resistance or relative density, the penetration resistance of silty sands is lower than that for clean sands. 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:

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

where ΔN_1 depends on the fines content of the silty sand. Tentative values of ΔN_1 are approximately as follows:

Fines Content	ΔN_1
$\leq$ 5%	0
$\simeq$ 15%	3
$\simeq$ 35%	5
$\simeq$ 50%	7

but judgement 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

1. 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 non-liquefied 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 1 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 non-liquefied soil near the toe and the crest and the residual strength of the liquefied sand. From the known strengths of the non-liquefied 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 700 to 750 psf. It may have been reduced as sliding progressed.

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 $(N_1)_{60} \simeq 15$ respectively. The $(N_1)_{60}$ value of about 15 is also indicated by SPT tests performed before the earthquake.

2. 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 8.

3. 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 1935 (U.S. Army Corps of Engineers, 1939; Casagrande, 1965). From the configuration of the slide material after failure, Bryant et al. (1983)

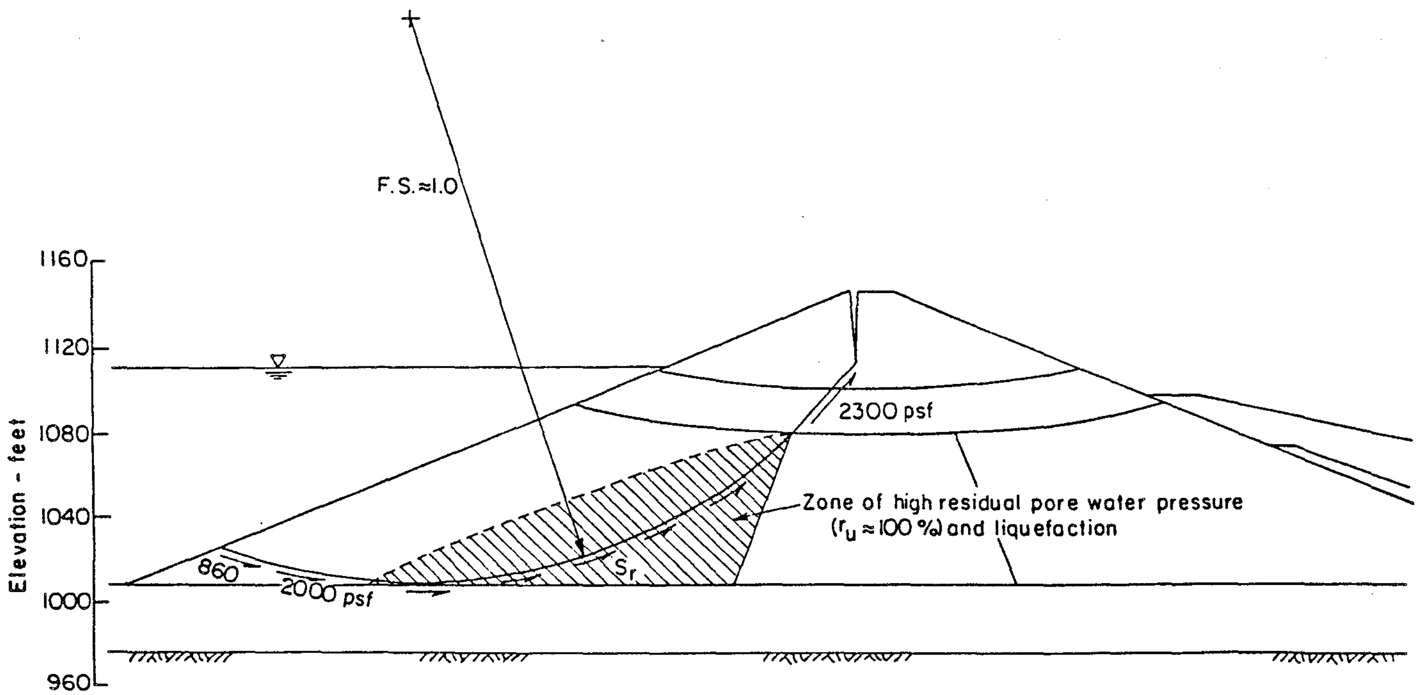

Figure 1. Cross-Section of Lower San Fernando Dam at End of Earthquake

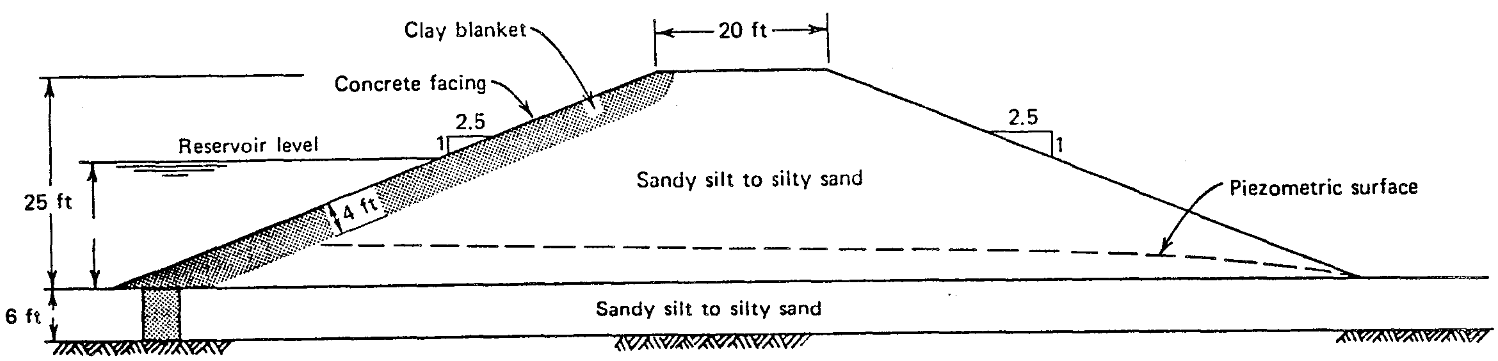

Figure 2. Cross-Section Through Sheffield Dam at Time of Failure

concluded that the residual strength of the liquefied sand was about 200 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 Krinitzky, 1976), led to the conclusion that the relative density of the sand was about 45%. This would correspond to a value of $(N_1)_{60}$ for a clean sand of about 11.

4. <u>Slide at Cape Lopez, Gabon</u>

A liquefaction slide occurred in a coastal deposit of sand at Cape Lopez, Gabon in 1971. Subsequently a similar deposit of sand was built up in the same location by similar geologic processes during the period 1972 to 1984, and the penetration resistance of this new deposit, which would be expected to be very similar to that of the original deposit was measured in 1984. It is estimated that the effective $(N_1)_{60}$ value of this deposit was about 13.

From the configuration of the soil mass at the time of sliding and assuming that the ground surface after the slide is closely representative of the slip surface, it can be computed that the average shear stress at the time of failure was about 600 psf. Thus the residual strength of the liquefied sand must have been somewhat lower than this value.

5. <u>Mochi-Koshi Tailings Dam Slide</u>

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), who studied this slide 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 2, but allowing for the fact that the tailings consisted of very fine-grained (silt size) particles, the equivalent $(N_1)_{60}$ value is about 7.

6. <u>Juvenile Hall Landslide, San Fernando</u>

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 3000 ft long moved laterally about 5 ft on a gentle slope of about 1.5^o. 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 $(N_1)_{60}) \simeq 7$.

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 the passive pressure acting on the end of the slide mass. Thus sliding could only occur apparently when the inertia forces induced by the earthquake motions were operative in the down-slope direction.

Analyzing this situation using a Newmark-type deformation analyses leads to the conclusion that the residual strength of the sandy silt must have been close to zero, otherwise surface displacements of the order of 5 ft could not have occurred.

7. <u>Snow River Bridge Slide Movements</u>

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 was again about 1 to 1 1/2°. The residual strength of the liquefied sand was again very low, at least over this range of surface movement.

8. Calaveras Dam Slide

A liquefaction-type slide occurred in the upstream shell of the Calaveras Dam as it approached a height of 200 ft in 1919 (Hazen, 1918). The dam was a hydraulic fill structure and it was subsequently re-constructed using rolled fill construction. From the configuration of the slide mass, the residual strength of the liquefied sand is estimated to be about 500 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 about 10.

9. 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 100 psf; however the residual strength was significantly less than this, say about 50 psf. The relative density of the sand foundation was measured to be about 32%; this would correspond to an $(N_1)_{60}$ value of about 5.

10. Slope Failures 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 is clear that the slide movements (about 150 ft) occurred after the earthquake motions had stopped. The residual strength of the slide mass has been estimated to be about 40 psf and the penetration resistance of the sand was found to be about $(N_1)_{60} \simeq 5$.

11. Uetsu Railway Embankment

A sand fill placed to serve as a 33 ft high railway embankment failed during the 1964 Niijata 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^{o} and came to rest at a slope angle of about 4^{o}. 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 4.

12. Kona Numa Railway Embankment

Another small railway embankment, 10 ft high, at Koda Numa, Japan failed during the 1968 Tokachi-Oki earthquake (Mushina and Kumura, 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^{o}. Lucia (1982) estimated that the residual strength of the liquefied sand was about 25 psf. No data is 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.

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 1. 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. Never-the-less 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 developed in liquefied sands and silty sands for other deposits.

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 consolidated-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 re-consolidation in the laboratory to the stress conditions existing in the field. In the proposed procedure the steady-state strength

Properties of Liquefied Sand and Silty Sands

Structure	Relative Density	Equivalent Clean Sand $(N_1)_{60}$	Residual Strength	Cause of Failure
Lower San Fernando Dam	≃ 50%	≃ 15	≃ 750 psf	Earthquake
Sheffield Dam	≃ 40%	≃ 8	≃ 50 psf	Earthquake
Fort Peck Dam	≃ 45%	≃ 11	≃ 200 psf	Construction
Cape Lopez, Gabon	≃ 50%	≃ 13	≃ 600 psf	Tidal effects
Mochi-koshi Tailings Dam	--	≃ 7	≃ 200 psf	Earthquake
Juvenile Hall Slide	--	≃ 7	≃ 25 psf	Earthquake
Snow River Bridge	--	≃ 7	≃ 25 psf	Earthquake
Calaveras Dam	≃ 45%	≃ 10	≃ 500 psf	Construction
Dike, Solfatara Canal	≃ 32%	≃ 5	≃ 50 psf	Earthquake
River Bank, Lake Merced	≃ 40%	≃ 5	≃ 40 psf	Earthquake
Uetsu Railway Embankment	--	≃ 4	≃ 35 psf	Earthquake
Koda Numa Railway Embankment	--	≃ 4	≃ 25 psf	Earthquake

Table 1. Residual Strengths of Liquefied Sands

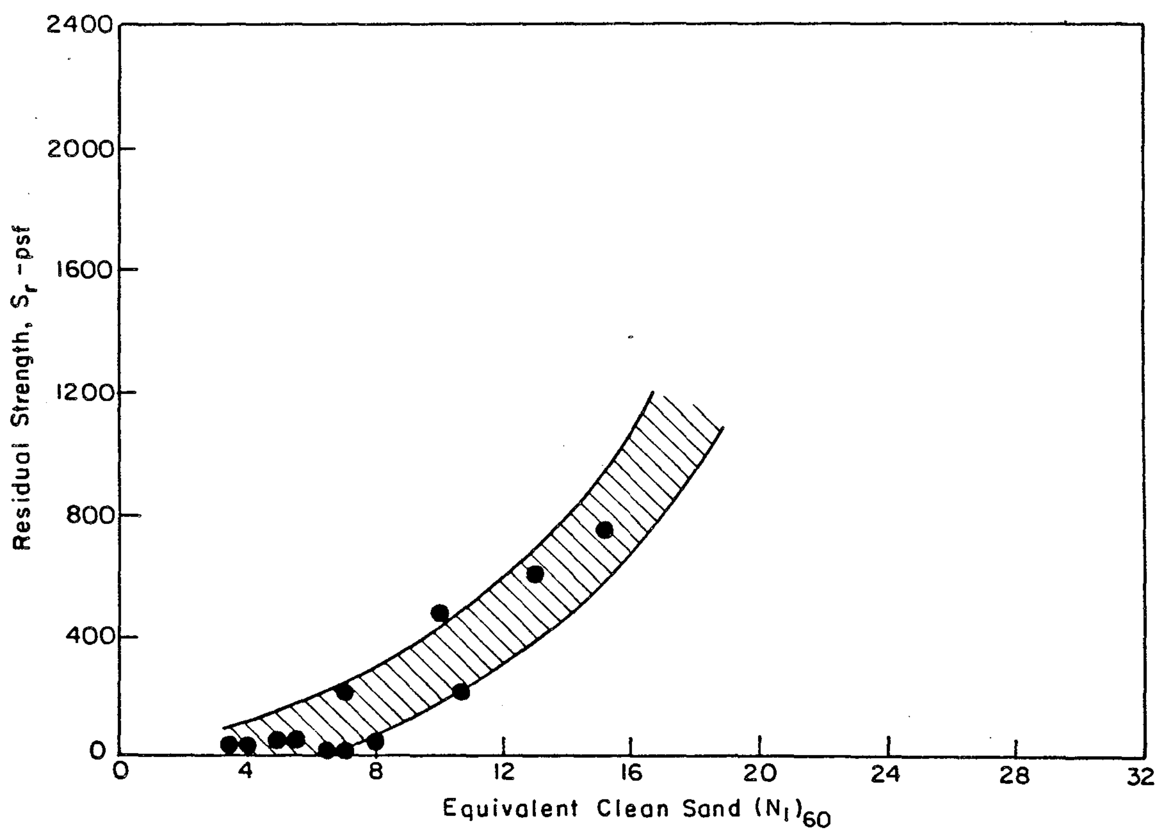

Figure 3. Tentative Relationship Between Residual Strength and SPT N-values for Sands (after Seed, 1984)

of a good quality undisturbed sample is determined at the laboratory void ratio after re-consolidation 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 re-constituted 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 re-constituted in the laboratory. Thus by performing tests on re-constituted 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 illustrated 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 seems to indicate that the procedure leads to significantly higher values of residual strength than those indicated in Fig. 3.

This may be due to the fact that a key assumption 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 re-distribution 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 Qiao, 1984) illustrated in 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

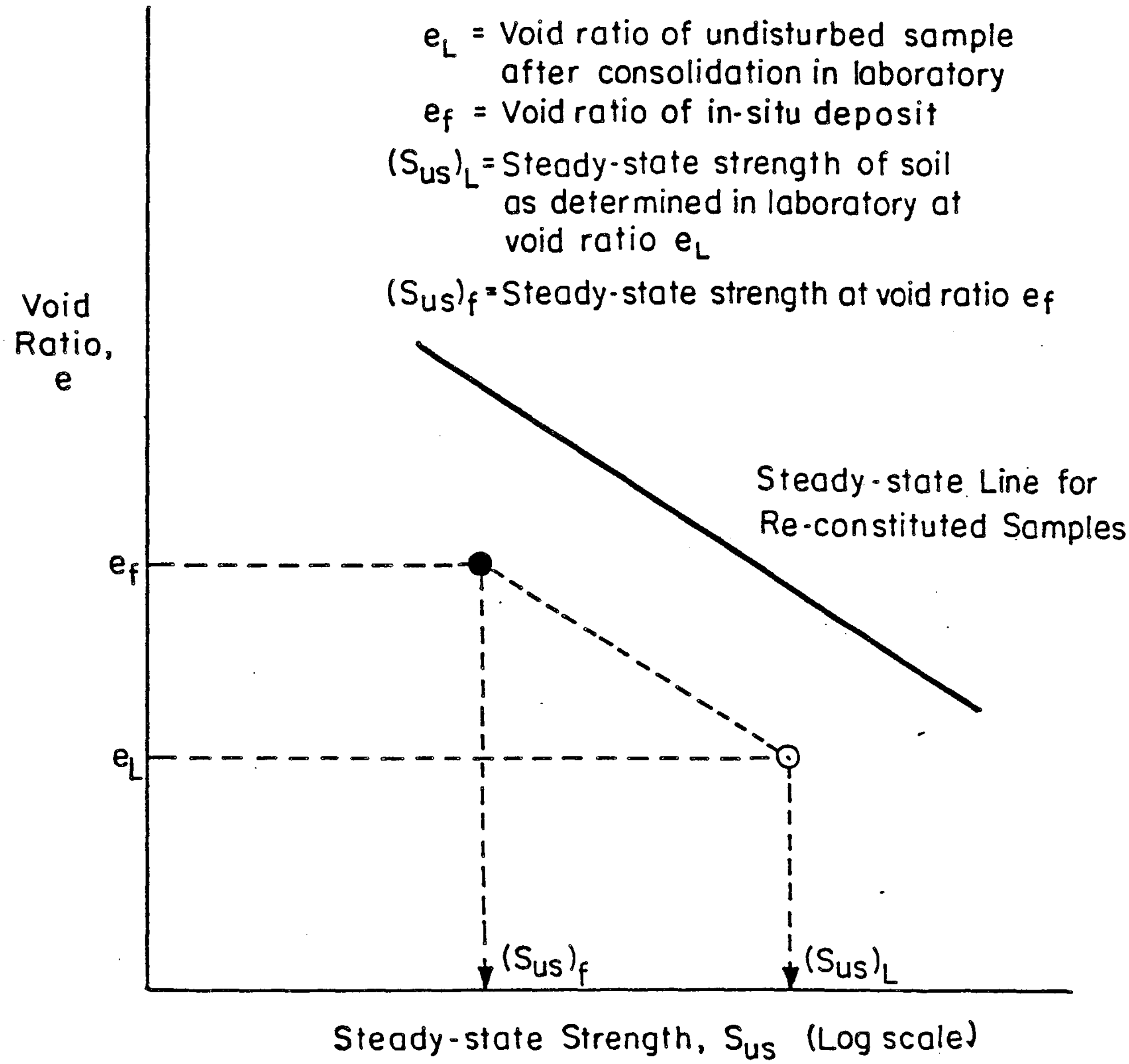

Figure 4. Procedure for Determining Steady-State Strength for Soil at Field Void Ratio Condition

re-distribution. The procedure by which this may occur has been described in a report by the NRC Committee on Earthquake Engineering (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 layer, where the void ratio near the end of earthquake shaking may be higher, and perhaps very much higher, than the initial (pre-earthquake) void ratio of the sand. Even if the validity of steady-state theory is accepted therefore (and the author 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 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

Test B-5 (all dimensions in cm)

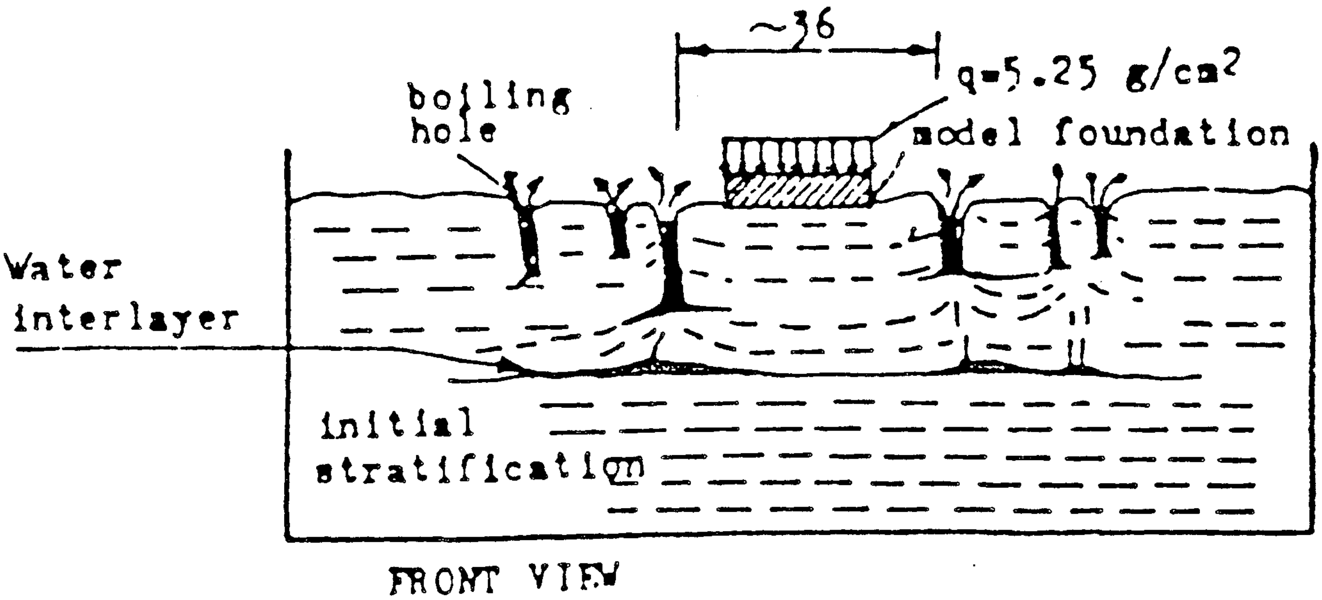

Figure 5. Results of Shaking Table Test on Deposit of Stratified Sand (after Liu and Qiao, 1984)

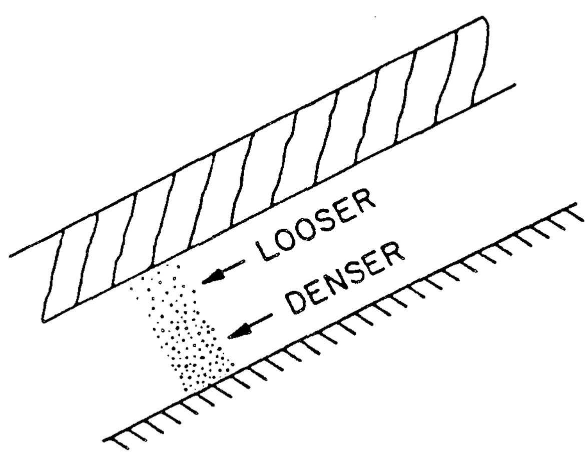

Figure 6. Example of a potential situation for mechanism B failure arising from the rearrangement of the soil into looser and denser zones. Local volume change occurs, but the sand as a whole remains at a constant volume and is "globally" undrained. (after NRC Committee on Earthquake Eng., 1985)

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 re-distribution 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 re-distribution in soil deposits subjected to earthquake shaking under undrained conditions. This problem does 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.

It may be noted 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 feet 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 to 2 degrees). 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 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 preceeding 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:

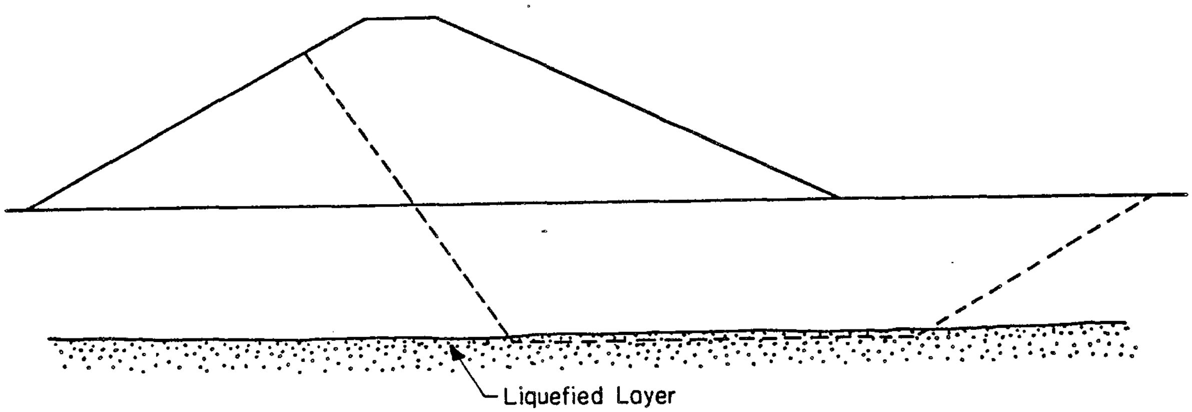

Figure 7. Schematic View of Low Embankment Underlain by Very Loose Sand Layer

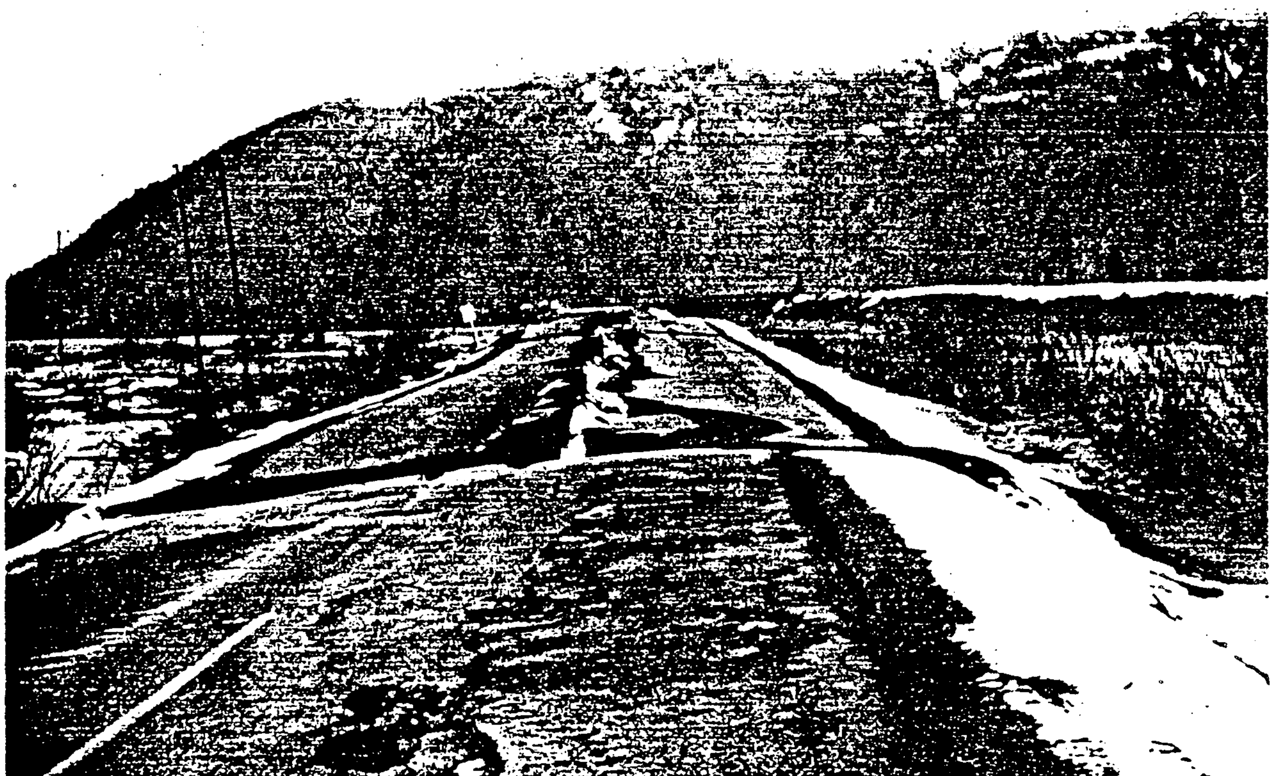

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Figure 8. Cracking of Embankment Associated with Lateral Spreading of Embankment in Alaska Earthquake (1984)

1. Deformations which occur due to liquefaction of a substantial body of soil part-way 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 stresses acting on the composite mass of liquefied and non-liquefied 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 re-distribution which may lead to failures or large deformations long after the earthquake motions have ceased (say 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 non-liquefied 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 non-liquefied 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 part-way through the period of earthquake shaking.

Thus when the computed factor of safety against the occurence 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 preceeding 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 high pore water pressures can not build up in the potentially liquefiable soil and thus liquefaction can not be triggered; 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 to simply ensure stability against major sliding after liquefaction has occurred; however evaluating this possibility requires a knowledge of the residual strength of the liquefied soil. It is suggested that water content re-distribution which has been observed in laboratory tests and may occur under field conditions makes this a difficult soil characteristic to determine by means of laboratory tests which do not permit water content re-distribution to occur, or by correction procedures which cannot anticipate the final field condition of the liquefied soil. Thus observations of the residual strength of liquefied soils in the field and the establishment of a relationship between this characteristic and some in-situ soil characteristic such as soil penetration resistance may provide the most practical method for evaluating residual strengths in problems where such values are required. Available data is summarized and plotted in chart form for this purpose.

Finally, the general principles of a design philosophy for handling liquefaction problems at the present state of knowledge is presented. It is suggested that the ability of the profession to predict the deformations of structures after liquefaction occurs in a substantial portion of the soil comprising or underlying a structure is not well enough developed at the present time to make this a suitable design methodology for critical structures. However where the risks associated with the possibility of large deformations occurring are considered acceptable, designing on the basis that such movements may occur may well provide an economically advantageous approach for the solution of design problems associated with soil liquefaction.

It is hoped that the results presented in this report 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 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.

Recognition of this basic principle is the key to successful practice in the field of geotechnical engineering. The principal has been stated at various times in different ways by different members of the engineering profession, but always with the same common concern with regard to the geotechnical engineer's responsibility for predicting the field performance of soil deposits and earth structures. It might well be termed General Principle A-1 concerning the application of new ideas, concepts and techniques in engineering practice. It is stated by Terzaghi and Peck in the following terms:

Terzaghi (1936): "No honest business man and no self-respecting scientist can be expected to put forth a new scheme or theory as a "working proposition" unless it is supported by at least fairly adequate evidence."

Peck (1978): "In soil mechanics, no evidence can be considered reasonably adequate until there is sufficient field

> experience to determine whether the phenomena observed in the laboratory are indeed the same as those that operate in the field. It must also be determined whether predictions based on laboratory studies are indeed fulfilled in the field . . ."

Engineers will necessarily have different opinions concerning the question of how much field experience is "sufficient" to validate any given concept or procedure, but in the light of past experience, there seems to be no basis for any disagreement over the need for some field validation of any new idea before it is applied in engineering practice.

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