LIQUEFACTION — THE STATE OF KNOWLEDGE

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This paper was presented as a keynote address at the Pacific Conference on Earthquake Engineering, Wairakei, 5-8 August 1987.

ABSTRACT

Although liquefaction of soils during earthquakes has been researched intensively for more than 20 years, it has remained a confusing problem - owing to seemingly divergent viewpoints as to the fundamental nature of the problem. During the past several years there has been a clarifying and coming together of these viewpoints, and hence a much clearer framework of understanding has been established. This new perspective is presented and related to previously developed methods of investigation and analysis, and remaining problems are identified and discussed. Several recent advances re parts of the problems - prediction of limited permanent deformations and delayed failure - are also summarized.

INTRODUCTION

The word "liquefaction" was thrust into the everyday vocabulary of earthquake engineering as a result of earthquakes in Japan and Alaska in 1964 and of the subsequent pioneering studies of H. Bolton Seed. Of course, it was quickly realized that the phenomenon was not new, and that failures attributable to liquefaction have occurred during almost every major earthquake documented in history.

Much research concerning "liquefaction" has been undertaken during the intervening years, considerable money has been spent on site investigations to ascertain susceptibility to liquefaction and upon remedial measures to eliminate or reduce the hazard, and countless words about the phenomenon appear in the literature. However, I think it safe to say that, at least until recently, there still has been considerable controversy, confusion and misunderstanding.

During the past few years, there has, I believe, been great progress toward reconciling viewpoints and clarifying key points. One very major step was preparation and issuance of the report Liquefaction of Soils During Earthquakes by the National Research Council. This report was prepared by the Committee on Earthquake Engineering of the National Academy of Engineering, based on a workshop (held at the Massachusetts Institute of Technology in March, 1985) with some 30 invited experts. The objectives of the study were to assess the state-of-the-art and to clarify and if possible reconcile conflicting viewpoints. The effort was partially successful in this latter goal, and stimulated further discourse (Whitman, 1985; Seed, 1987; Castro, 1987) since then. While there still remain strongly divergent viewpoints concerning methods for evaluating resistance to liquefaction, there is now substantial agreement on some key fundamental points.

The major part of this paper will set forth my own perspective concerning the state-of-the-art. In the latter portion, I will report upon some recent centrifuge-based experimental and theoretical results.

WHAT IS LIQUEFACTION?

Liquefaction is best understood as encompassing several different phenomena, all related to the tendency of saturated sands to experience decrease in resistance and build-up of pore pressures as a result of cyclic straining. Four different aspects of liquefaction are depicted in Figures 1 through 4, a classification suggested by Youd (1984).

Flow failures: Flow slides are dramatic expressions of liquefaction. Natural slopes, earth dams for water retention and mine waste tailing dams have all experiences such failures. In several instances, the failure occurred minutes or hours after the causative earthquake.

Bearing capacity failures: There have been numerous instances, primarily in Japan but elsewhere as well, of structures tilting and/or sinking into the sand as a result of an earthquake. Such failures are typically accompanied by evidence of upward flow of water, but clearly the mechanism of failure is a loss of bearing capacity. A related type of failure is the rising up of empty or partially empty buried tanks and pipes.

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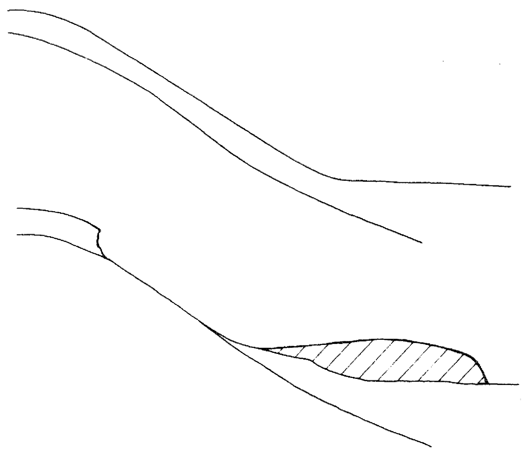

FIG. 1 - DIAGRAM OF A FLOW FAILURE. LIQUEFACTION DEVELOPS BENEATH THE GROUND SURFACE, CAUSING THE SOIL TO LOSE STRENGTH AND FLOW DOWN THE STEEP SLOPE (YOUD, 1984)

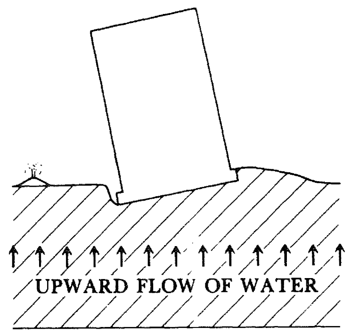

FIG. 2 - TILTING OF A BUILDING FROM LIQUEFACTION AND LOSS OF BEARING STRENGTH IN SOIL (YOUD, 1984)

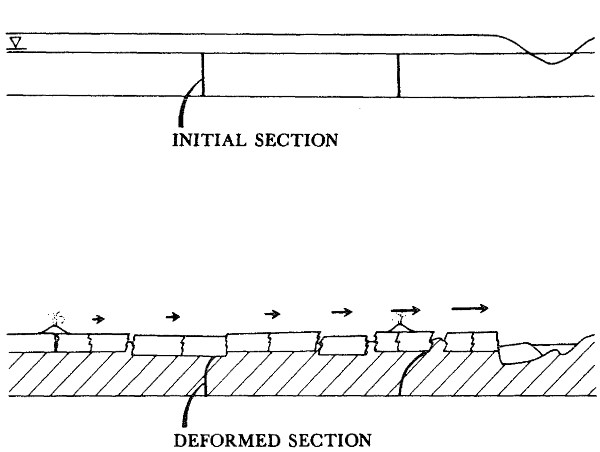

FIG. 3 - DIAGRAM OF LATERAL SPREAD BEFORE AND AFTER FAILURE. LIQUEFACTION OCCURS IN THE CROSS-HATCHED ZONE. THE SURFACE LAYER MOVES LATERALLY DOWN THE MILD SLOPE, BREAKING UP INTO BLOCKS BOUNDED BY FISSURES (YOUD, 1984)

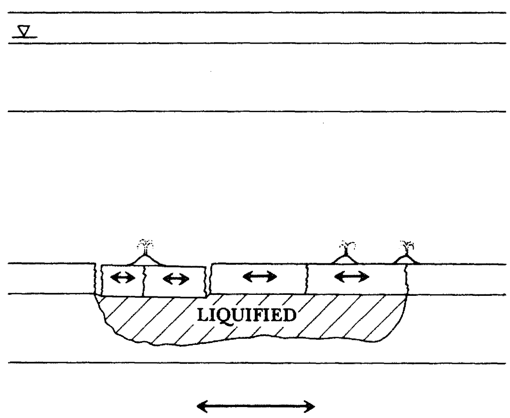

FIG. 4 - THE MECHANISM OF GROUND OSCILLATION. LIQUEFACTION OCCURS IN THE CROSS-HATCHED ZONE, DECOUPLING THE SURFACE LAYER FROM THE UNDERLYING FIRM GROUND. FISSURES FORM AND IMPACTS OCCUR BETWEEN OSCILLATING BLOCKS AND ADJACENT FIRM GROUND (YOUD, 1984)

Lateral Spreading: This phenomenon, often referred to in the literature as lurching, involves lateral movement of soil on slopes as gentle as 1 or 2 degrees. It is in effect a form of slope failure on a slope so gentle as to appear as level ground. A classic example is the lateral spreading causing failure of the Juvenile Hall in San Fernando, California, in 1971.

Ground Oscillation: Here loss of shear resistance in a layer of soil means that ground motions coming from below cannot be transmitted through the sand to ground surface. Hence relative displacements develop between the surface and some depth accompanied by a breaking up of the surface with differential motions between blocks. Sand boils often accompany this aspect of the problem.

Flow failures and bearing capacity failures are the most eye-catching problems, and hence have captured the attention of both laymen and engineers. However, lateral spreading and excessive ground oscillation can cause considerable damage - to pipelines, roadways, the ground floor slabs of light buildings, etc. Where sand boils occur at paved areas or within buildings the cost of clean-up can be considerable. Youd has claimed that the property losses from lateral spreading and ground oscillation have, in earthquakes of this century, exceeded those from flow and bearing capacity failures.

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Observation during earthquakes of these phenomena has prompted extensive laboratory investigations, primarily involving cyclic loading of samples in triaxial or simple shear tests. It has been well established that:

* Excess pore pressures develop, even in dense sands, during cyclic straining with undrained conditions.
* The development of such excess pore pressures is related to the tendency of all sands to densify during drained cyclic straining.
* If pore pressures build up to the point where effective stresses become zero or very small during any time within a cycle of loading, large cyclic strains usually develop.
* The tendency to build up pore pressures, and any associated tendency to develop large cyclic strains, increases as the relative density of the sand decreases.
* If sand is loose enough, application of cyclic stress in addition to a sustained shear stress can cause complete collapse of a test specimen.

Much else has been learned from this research, such as the sensitivity of the results to sample disturbance or the manner in which a sample is reconstituted, and to test conditions.

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During the past two decades, there has been much controversy as to whether the word "liquefaction" should be applied to all of the phenomena described above. Various other phases - cyclic mobility, initial liquefaction with limited strain potential - have been suggested as being more apt for one or more of the occurrences, but their usage has not taken hold. The engineering profession, worldwide, has come to use "liquefaction" indiscriminately for any occurrence involving evidence of high pore pressures and/or loss of resistance in cohesionless soils. I use the word in this general sense.

However, some distinctions and definitions clearly are necessary. In particular, it is necessary to distinguish between: (a) flow failures and deformation failures, and (b) cases where the static equilibrium soil must or need not sustain shear stresses.

Flow failures involve movements of soil over considerable distances, with gross change in the geometry of the earthen mass. Deformation failures involve the development of permanent movements large enough to require remedial measures or at least investigations to establish the cause of the movements and the possible need for corrective actions. The definition of deformation failures is somewhat vague, and there are those who prefer not to use the word "failure" in connection with such happenings. However, such excessive deformations certainly are failures in the eyes of clients and the public and hence the engineers.

In a slope, it is necessary that the earth be able to sustain shear stresses before, during and after an earthquake. Otherwise a flow failure will certainly occur. This is also true of soil providing bearing for a building, and in many other geotechnical engineering problems. On the other hand, beneath level ground the soil need not be able to sustain shear stresses in order that there be static equilibrium. If the shear stresses associated with a $K_o \neq 1$ condition were to disappear, there would not be a loss of equilibrium.

The following discussion focuses first upon flow failures, which inherently means situations in which shear stresses are required for static equilibrium. Later the level ground case and deformation failures will be considered.

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Primarily for convenience, this paper will talk about liquefaction in terms of sand. The phenomena can also occur in nonplastic silts. The presence of plastic fines does have an inhibiting influence, although similar (and probably related) phenomena do occur in clays of low plasticity. The larger permeability associated with coarser soils makes it less likely there will be the undrained conditions that favour liquefaction, but liquefaction has been observed in gravelly soils.

CASES INVOLVING SUSTAINED SHEAR STRESSES

Figure 5 is a sketch of a slope and a failure surface. A shear stress, generally called the driving shear stress, exists along this surface. Initially, there must be a resisting shear stress at least equal to the driving stress; so as to provide equilibrium for the overlying mass of earth. Assuming that this is the critical failure surface, a flow slide can occur only if - as the result of an earthquake - the shear strength along the surface decreases to less than the driving stress.

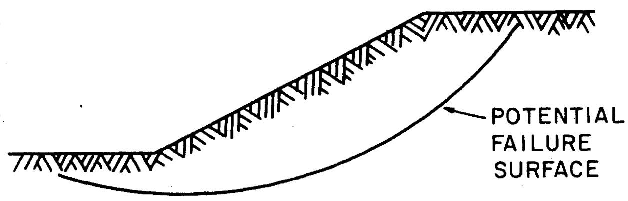

FIG. 5 - SLOPE WITH POTENTIAL FAILURE SURFACE

A type of stress-strain behaviour that can lead to a flow failure is sketched in Figure 6. Such a curve is observed during undrained monotonic straining of loose sand. Cyclic stresses superimposed upon the initial stress may take the soil past the peak of the curve whereupon its strength decreases to the final steady state undrained strength, S_{us}.

Conversely, a medium dense-to-dense sand has a stress-strain curve during monotonic undrained straining that rises steadily to its final value (Figure 7). In such a soil, the undrained steady state strength must be greater than any initial sustained shear stress. It follows that with such a soil a flow failure cannot occur under undrained conditions.

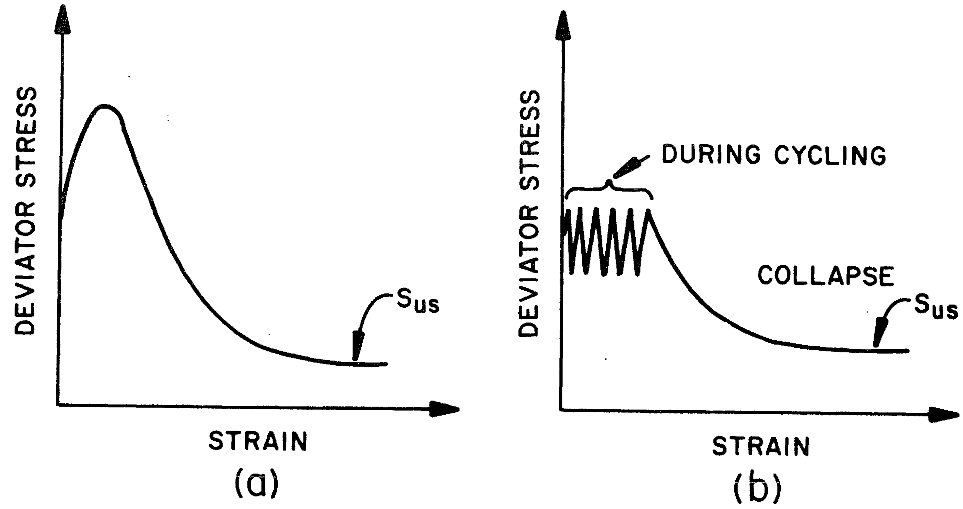

FIG. 6 - STRESS-STRAIN BEHAVIOUR FOR UNDRAINED LOADING OF LOOSE SAND

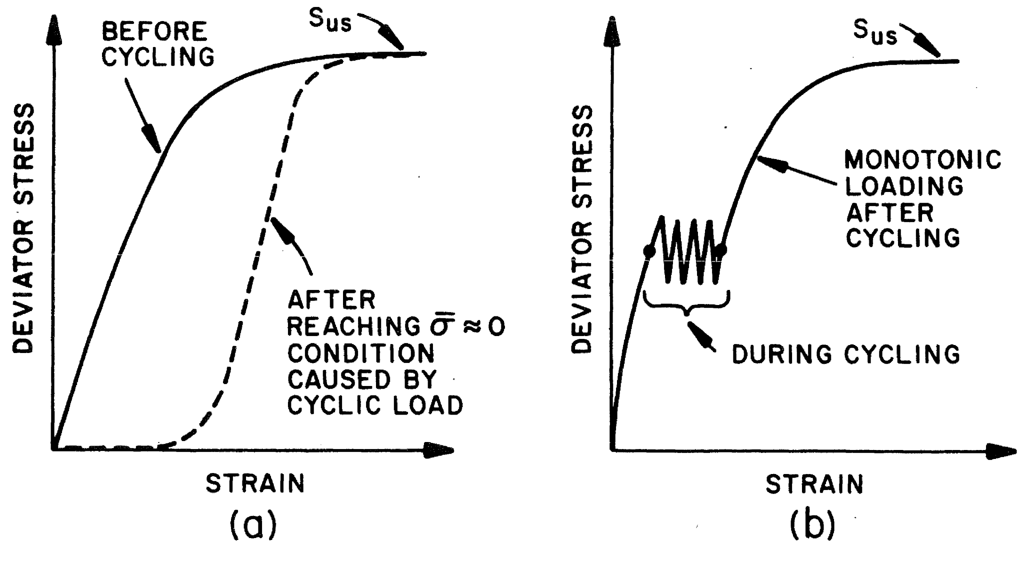

FIG. 7 - STRESS-STRAIN BEHAVIOUR FOR UNDRAINED LOADING OF DENSE SAND

If a mass of saturated sand remains in a constant volume condition at each and every point during an earthquake, deciding whether or not there can be a flow slide involves no more than determining whether the sand has the type of stress-strain behaviour sketched in Figure 6 and whether S_{us} is less than the driving stress. This is the viewpoint that has been put forth by Castro and his colleagues. However, this still leaves a question as to whether a flow slide might develop in a medium dense-to-dense sand because of departures from truly undrained conditions.

One possible circumstance is sketched in Figure 8. If the inclined sand seam experiences cyclic straining, there is a tendency for the loosened sand particles to settle, so that the lower part of the seam becomes denser while the upper part loosens. Even though the sand seam is undrained overall, locally volume changes occur. In this way it is possible that the undrained steady state strength of the upper portion of the sand is reduced.

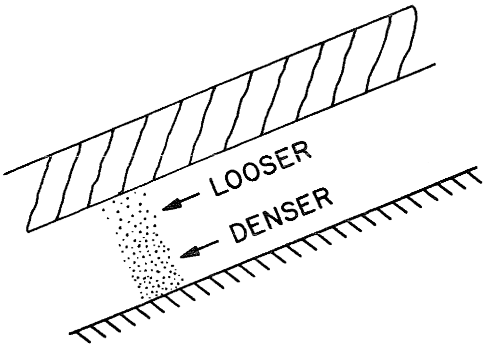

FIG. 8 - POSSIBLE CHANGES IN VOLUME WITHIN A STRATUM FOLLOWING SHAKING

Another possibility is shown in Figure 9. As the pore pressures generated in the sandy zone push upward toward the slope, the overlying soil is cracked and weakened by increasing pore pressures, and the outward flow of water may loosen the upper part of the sand. These changes may cause the strength along a failure surface passing through both sand and overlying soil to decrease below a value required for equilibrium.

These two possibilities involving departures from undrained conditions are speculative. The situation depicted in Figure 7 might well account for the spectacular slide in varied material at Riñihue during the 1960 earthquake in Chile. Something like these possibilities must be present to explain the delay in some slope failures following the cessation of ground shaking.

For further reference, note that Castro's viewpoint implies that, as the initial sustained shear stress increases, it should require smaller cyclic stresses to cause a flow slide. That is, the closer one is to the peak initially, the easier it is to push the soil over the hump.

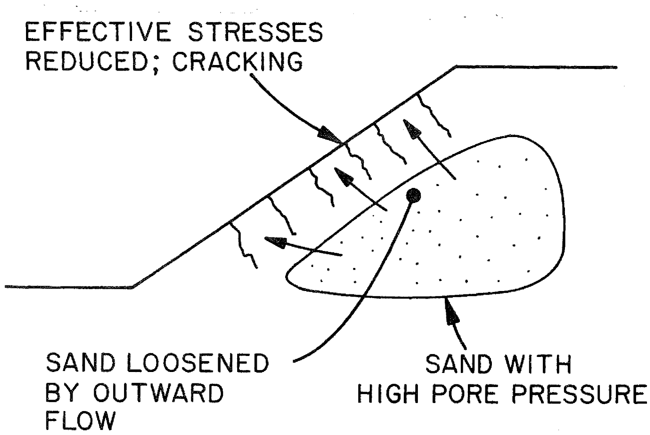

FIG. 9 - REDUCTION OF OVERALL STRENGTH FROM OUTWARD MOVEMENT OVER PORE WATER

Evaluation of Available Strength

There is now, I believe, general agreement upon the importance - when assessing the stability of slopes involving cohesionless soils during and after earthquakes - of the strength available following cyclic straining. This is a significant step forward. However, there is still a major disagreement as to how best to evaluate this strength and as to the possible role of departures from truly undrained conditions.

Poulos et al. (1985) have evolved a procedure for evaluating S_{us} using undisturbed sampling. Sampling is done with a fixed piston sampler, and the length of the sampler is measured immediately upon removal of the tube from the ground. Undrained steady-state strength is measured using undrained triaxial tests with monotonic straining, and the measured S_{us} is then corrected for void ratio changes during sampling, transportation of samples and preparation of specimens for testing. The correction is based upon the slope of an S_{us} vs. void ratio curve determined from tests upon reconstituted samples. The corrections can be large: factors of 4 to 8 are common. As yet there have been but a few case studies involving actual flow slides by which this procedure may be judged; one such recent study has been a re-evaluation of the slide at the Lower San Fernando dam during the earthquake of 1971 (Castro et al., 1987).

Seed (1984) has assembled a series of case studies in which he has related residual strength, as deduced from flow slide failures, to corrected standard penetration resistance. This correlation is reproduced in Figure 10. Residual strength is similar to undrained steady state strength, except that, being empirically determined from flow slides, it may reflect local changes in void ratio. As is evident in the figure there is considerable scatter in the data.

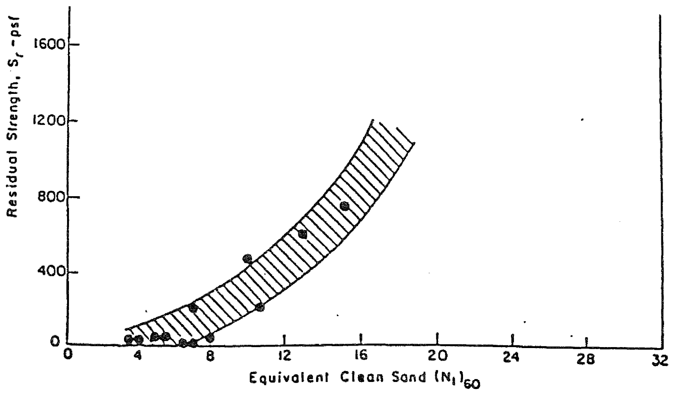

FIG. 10 - TENTATIVE RELATIONSHIP BETWEEN RESIDUAL STRENGTH AND SPT N-VALUES FOR SANDS (AFTER SEED, 1984)

Thus, while it is now agreed that S_{us} or residual strength is a key parameter, it cannot now be evaluated with confidence. If a conservatively low value of strength is used and a slope is found to be stable, then earthquakes cannot trigger a flow slide. Conversely, if, with a conservatively high value, a slope has a safety factor less than unity, a flow slide can be triggered by strong enough ground shaking. Intermediate situations will continue to tax the skill and judgement of engineers.

Needed Research

There is an obvious need for many more case studies from which S_{us}/residual strength is evaluated and compared to proposed methods for evaluating this strength. This need presents a major challenge for the future.

There is also need to be able to predict the intensity of ground shaking required to trigger a flow slide, given that S_{us}/residual strength is small enough to allow a flow slide to occur. There has been excellent research upon this problem by Dobry et al. (1984).

Finally, there is a need to understand and evaluate the possible role of departures from constant volume conditions. Tests upon elements of soil are of little help with this problem. Theory can be useful as an indicator of what might happen, but theory alone will not put the matter at rest. Since opportunities to make detailed observations and measurements upon full-scale earth structures are scarce, there is a great potential role for model tests.

As an aside, mentioning model tests gives me an opportunity to talk about another favourite subject: centrifugal model testing. By now the basic principles of centrifugal modelling have been discussed thoroughly (e.g. Schofield, 1981). The basic principle is to achieve self-weight stresses similar to those encountered in full-scale situations. There are various difficulties about the scaling of time and permeability, but a number of studies of liquefaction phenomena have been made (Heidaro and James, 1982; Lambe and Whitman, 1981; and others).

A recent series of tests at MIT involved the quick-release of a wall retaining a saturated sand backfill (Pahwa, 1987). The test arrangement is shown in Figure 11. The counterweight, connected to the top of the wall by cable and pulleys, allows the static safety factor to be adjusted to preselected values less than unity. The strut, which initially provides the additional support necessary for equilibrium, is suddenly removed and the wall begins to fall. Most tests involved a dense sand as backfill, and negative excess pore pressures were

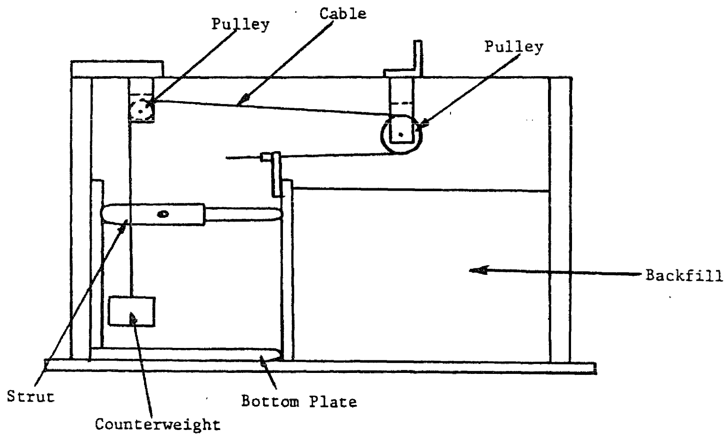

FIG. 11 - APPARATUS FOR QUICK RELEASE OF WALL RETAINING SATURATED SAND.

observed to occur. These reduced pore pressures caused retardation of failure until they dissipated.

Of greatest interest here was one test

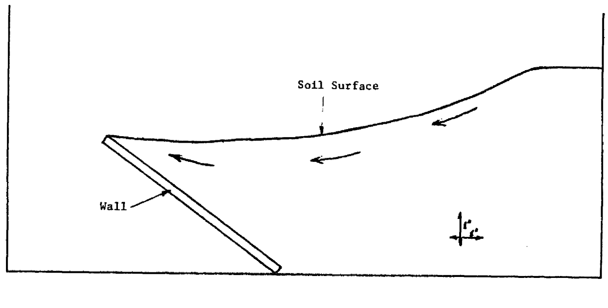

FIG. 12 - FLOW OF LOOSE SAND FOLLOWING SUDDEN RELEASE OF RETAINING WALL

with a very loose saturated sand as backfill. Upon removal of the strut, the wall immediately collapsed totally and the sand was observed to flow out with the wall. Indeed, when motion of the wall was arrested by stops, the sand flowed on over the top of the wall (Figure 12). Clearly it is possible to have liquefaction flow failures on a centrifuge!

LEVEL GROUND

The foregoing discussion has emphasized undrained steady state strength and residual strength, with very little mention of buildup of excess pore pressures. At this point you may wonder: What has happened to the notion - that liquefaction occurs when pore pressures build up to equal total stress and effective stress is reduced to zero - that has dominated discussions of liquefaction for two decades? The answer is: the concept of liquefaction as a condition of $\sigma' \approx 0$ applies to level ground (Seed, 1976) - or more particularly to cases where it is not necessary to have shear stresses in order for static equilibrium.

In order to emphasize the important distinction, consider the effective stress paths in two simple shear tests with cyclic loading: see Figure 13. (Only average stress paths are shown; the fluctuating stress paths during each cycle have been omitted for clarity.) Part (a) of the figure shows the behaviour when there is a sustained shear stress; different stress paths develop as a the failure line is approached, depending upon the density of the sand, with with a loose sand a flow failure is possible without the effective stress ever dropping to zero. Part (b) represents the case where there is no sustained shear stress. The initial shear stress arises because of $K_o < 1$ consolidation, but this stress disappears as cyclic stressing continues and for all soils the average stress path approaches the origin and $\sigma' \approx 0$.

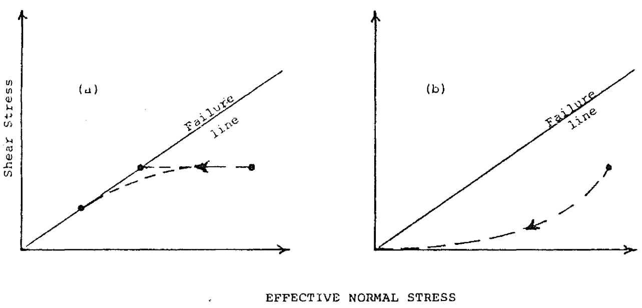

FIG. 13 - AVERAGE EFFECTIVE STRESS PATHS DURING CYCLIC LOADING: (a) WITH SUSTAINED SHEAR STRESS, (b) WITH NO SUSTAINED SHEAR STRESS

There has been little disagreement about the fundamentals of the level ground case, although there are different viewpoints as to the practical importance of such situations. However, as pointed out earlier, a large amount of damage to pavements, pipelines, etc., can occur because of excessive ground oscillation and the development of sand boils.

A number of different approaches to evaluation of level ground sites have been proposed and utilized. Much has been written on this subject, and it is not my purpose to review all of it. A very thorough discussion appears in the aforementioned report from the U.S. National Research Council. I shall mention briefly only a few highlights and offer several opinions.

The state-of-the-art in the United States continues to be use of the standard penetration test plus charts derived from numerous cases of liquefaction or nonliquefaction. A recent version of such a chart is reproduced in Figure 14. The distinction between liquefaction or nonliquefaction is generally whether or not sand boils were observed. While clearly empirical, the testing done in laboratories plus simple theoretical considerations influenced the form of the normalized parameter plotted vertically. The blow count from the standard penetration test (SPT) is corrected so as to apply to a depth where the overburden stress is 1 ton/ft^2 and to a 60% energy ratio (actual to theoretical energy delivered to the drill rod). Separate curves have been developed for different contents of fines in the sand.

One question associated with this approach is just where should one draw the curve separating liquefaction and nonliquefaction and how far below the curve should the point representing a site be for the site to be safe against liquefaction. These questions have recently been addressed in a statistical study by Liao (1986), who determined in a systematic way the location of a curve corresponding to 50% probability of liquefaction (and for smaller probabilities as well). He also ascertained whether certain factors - such as whether blow counts were measured before or after the earthquake - had a statistically significant influence upon the location of the curve for a given probability. The content of fines was indeed found to have a statistically significant effect, increasing the resistance to liquefaction for a given corrected blow count, but the statistically justifiable increase was not as large as that indicated on Seed's charts.

Most engineers actually feel that cone penetration resistance (CPT) should be a better tool for site investigation than SPT. However, as yet there is still not an adequate data base to permit direct construction of a diagram, with cone resistance as the parameter, such as that in Figure 14 based upon SPT. Correlations between blow count and cone resistance are used, but this introduces additional uncertainty. I believe that use of CPT will be the way of the future, but for the

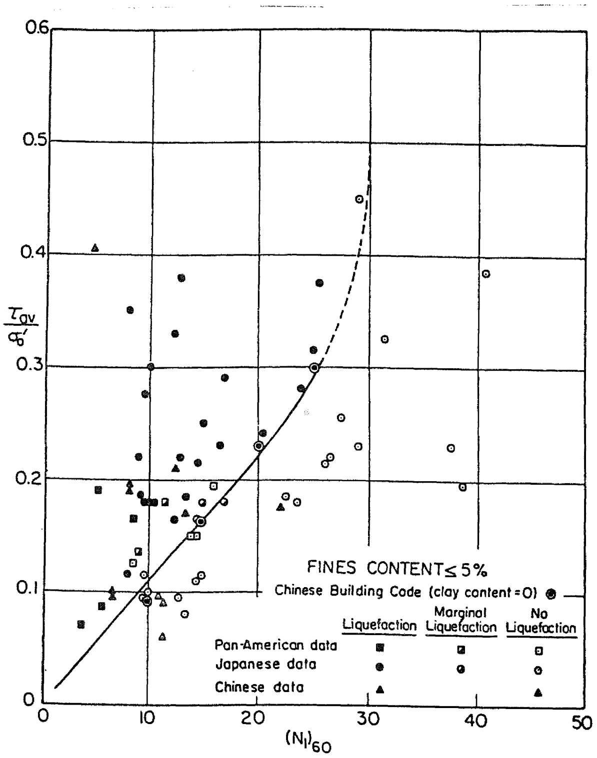

FIG. 14 - RELATIONSHIP BETWEEN STRESS RATIOS CAUSING LIQUEFACTION AND $(N_1)_{60}$ VALUES FOR CLEAN SANDS FOR MAGNITUDE 7.5 EARTHQUAKES. (SEED el al., 1984)

present use of the SPT is still standard.

Application of Level Ground Analysis to Slopes

Some years ago, Seed, et al. (1975) described how the above-discussed analysis for liquefaction of level ground might be applied to soils within or beneath slopes. A correction was applied to resistance determined from a chart such as Figure 13, to account for the influence of having a sustained shear stress. This correction was obtained comparing, using cyclic triaxial tests with and without sustained deviatoric stress, the combinations of cyclic stress amplitude and number of cycles required to cause some selected cyclic strain - usually 5%. The proposed correction was such that having a sustained shear stress increased the resistance to liquefaction.

Given the distinction that has been made between the case of slopes and level ground, and recalling the earlier conclusion that increasing the sustained shear stress should make a flow slide more likely, what can now be made of an analysis of this type? An associated question is: What may happen if an embankment or structure is constructed upon ground which may experience level ground liquefaction? The following discussion focuses upon sands of medium density, which is the case where the argument is most significant.

Before answering the question, it is necessary to understand why triaxial test results indicate that having a sustained deviator stress is beneficial. This may be done by contrasting the two sets of results shown in Figure 15. In part (a), the effective stress path moves to the left as pore pressures increase during cycling, but when the effective stress path reaches the failure envelope it stabilizes. In this situation, the cyclic strains typically are small. In part (b), on the other hand, pore pressures continue to increase after the effective stress path first reaches the failure envelope, the stress path tips so as to lie close to the failure line, and the minimum effective stress during part of a cycle is close to zero. In this second case, quite large cyclic strains usually develop.

The reason for this difference in behaviour is that in the second case there is zero applied deviator stress at two times in each cycle, associated with reversal in principal stress directions. It appears that sand particles are most free to rearrange, leading to a tendency to densify and to greater potential for straining, if there are moments when there is no applied shear stress. Thus the observed effect of sustained shear stress is explained: As the sustained shear stress increases, the cyclic stress must increase for there to be stress reversals.

In neither of the tests reported in Figure 15 was there complete collapse; the undrained steady-state strength for these samples was greater than the maximum applied stress. Hence, to put the earlier question

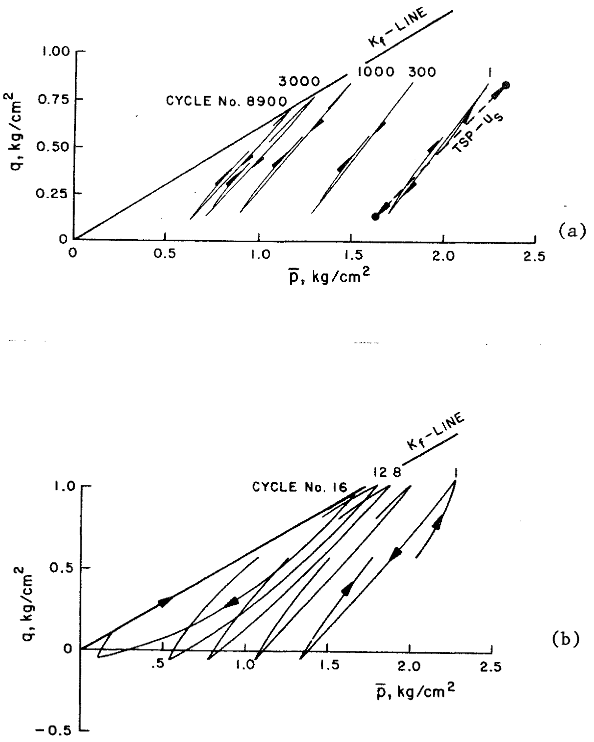

FIG. 15 - EFFECTIVE STRESS PATHS FROM CYCLIC TRIAXIAL TESTS ON SATURATED SAND WITH RELATIVE DENSITY ≈ 42%: (a) DEVIATOR STRESS ALWAYS COMPRESSIVE; (b) DEVIATOR STRESS REVERSES DIRECTION. (HEDBERG, 1977).

in another way, what do results from tests such as those of Figure 15 have to do with the safety of slopes? There are two responses.

If a level-ground-plus-correction-for-sustained-shear-analysis shows that excessive cyclic strains will not develop, then most assuredly there is no danger of a flow failure. Thus, use of this relatively simple approach may make it unnecessary to undertake the difficult sampling and testing necessary to obtain a satisfactory evaluation of S_{us}.

On the other hand, a prediction of large cyclic strains does not necessarily imply there will be a flow failure or slope instability. Evaluation of residual strength then becomes necessary. However, a prediction of large cyclic strains is a tip-off that there may at least be significant permanent deformations - which brings us to the final topic.

DEFORMATION FAILURES

Analysis of permanent deformations of earth structures as a result of seismic shaking or other cyclic or transient loadings is one of the major present-day frontiers of soil dynamics. These problems have helped spawn the recent and current interest in constitutuve modelling. This is potentially a very large subject in its own right, and discussions here will be limited to approximate solutions for three situations.

Lateral Spreading of Elasto-Plastic Sand

Castro (1987) has recently suggested that the amount of seismically-induced lateral spreading of gentle slopes in saturated sands can be predicted using the Newmark sliding-block analogy (Newmark, 1965). In such situations the sustained shear stress is small, and the steady-state undrained shear strength is generally larger than this driving stress so that no flow failure can develop. However, if the peak dynamic plus static shear stress reaches S_{us}, then an increment of lateral sliding will occur. Castro analyzes a case study of lateral spreading in this manner.

This is a sound and interesting concept and deserves further study. There may well be a peak to the sand's stress-strain curve that must be overcome before plastic behaviour begins, and the influence of this initial peak must be ascertained. The range of conditions to which this approach might apply - which appears to involve slopes inclined at 5 degrees and less with loose sand - must also be determined.

Slopes Containing Pockets with Large Potential for Straining

This is the type of situation depicted in Figure 9. If the sand in the pocket develops, as a result of cyclic straining, a potential for large permanent shear strains, then the static stresses must redistribute within the slope causing a permanent distortion of the earthen mass.

Any situation such as this, where there are zones with potentially very different stiffnesses, is difficult to analyze by any sophisticated constitutive model. Approximate methods have been suggested by Seed et al. (1975) and by Marr and Christian (1981). In these very similar approaches, one effectively determines a reduced shear modulus for the portion of the soil that softens, using the permanent shear strains that are predicted to occur for the specified cyclic loading. This reduced modulus is then used in a static finite element calculation that redistributes the gravity loading through the earthen mass.

Soils with Moderate-to-Small Potential for Permanent Strains

There are a number of practical problems that potentially fit this description, such as buildings founded over saturated sands where even a few inches of settlement might be damaging. Clearly there is a need to be able to predict the magnitude of settlement or permanent deformation that might occur, but also a need to have actual data against which the validity of such predictions may be

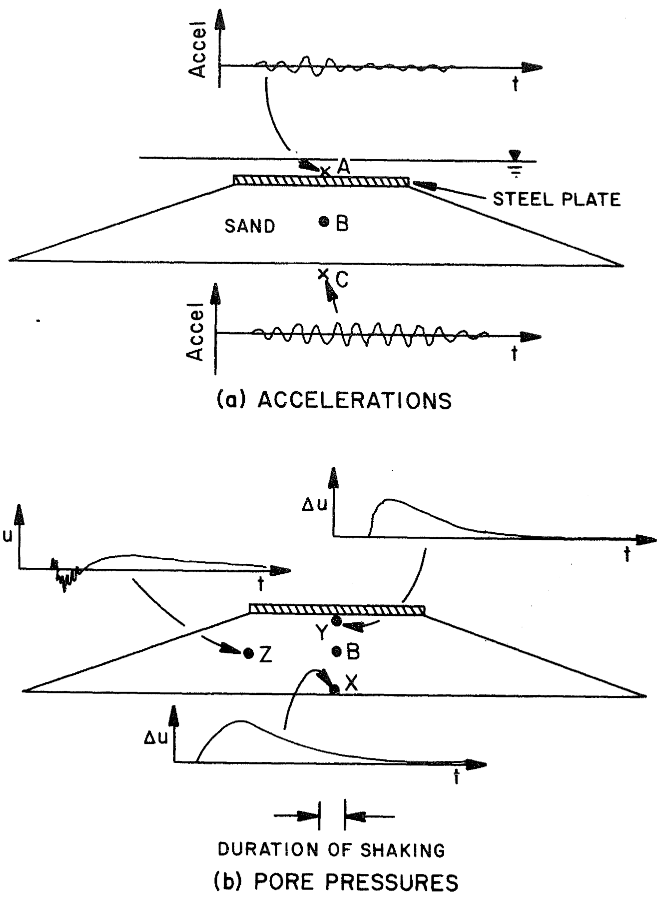

FIG. 16 - ACCELERATIONS AND EXCESS PORE PRESSURE DURING SHAKING OF SAND ISLAND ON CENTRIFUGE. THE VERTICAL SCALES DIFFER AMONG THE VARIOUS PLOTS. (ADAPTED FROM LEE AND SCHOFIELD, 1984).

checked. Since observations during actual earthquakes are so sparse, we return to experiments carried out on a centrifuge. The particular experiments I will discuss were carried out on the centrifuge at Cambridge University, using the "bumpy road" excitation that subjects the model to ten more or less sinusoidal cycles of shaking at its base.

Figure 16 shows accelerations and excess pore pressures in an experiment that models an off-shore platform resting upon a submerged sand island (Finn, et al., 1984). There is clear evidence that a liquefaction occurs just below the steel plate that simulates the structure: high excess pore pressures develop here and after a few cycles the applied base acceleration can no longer be transmitted up to the plate. Note also the excess pore pressure at point Z; after an initial rise the excess pore pressure becomes negative for a time. One has the impression that the slope has begun to fail and hence the soil dilates. Finn has used these results to check and improve his computational model TARA.

Figure 17 shows another experiment in which a structure - consisting of lead shot within a frame and separated from a stratum of saturated sand by a horizontal flexible membrane - is shaken. The lower part of the figure shows the location of pore pressure transducers. Figure 18 reproduces plots of excess pore pressure vs time at several locations. While there are no scales on these plots, they have the same scales and pore pressures equal to the total overburden stress occur at the two locations at the right of the figure. That is, a liquefaction occurred in the free field away from the structure.

These results have been compared to predictions made after-the-fact by Stamatopoulos and Whitman (1987) using a theory reported by Bouckovalas et al. (1984). In this approximate theory, dynamic elastic analysis is used to determine cyclic stresses. For each element of a finite element grid, the permanent strains resulting from one or more cycles of such stresses - as applied in drained triaxial tests - are evaluated. The nodal

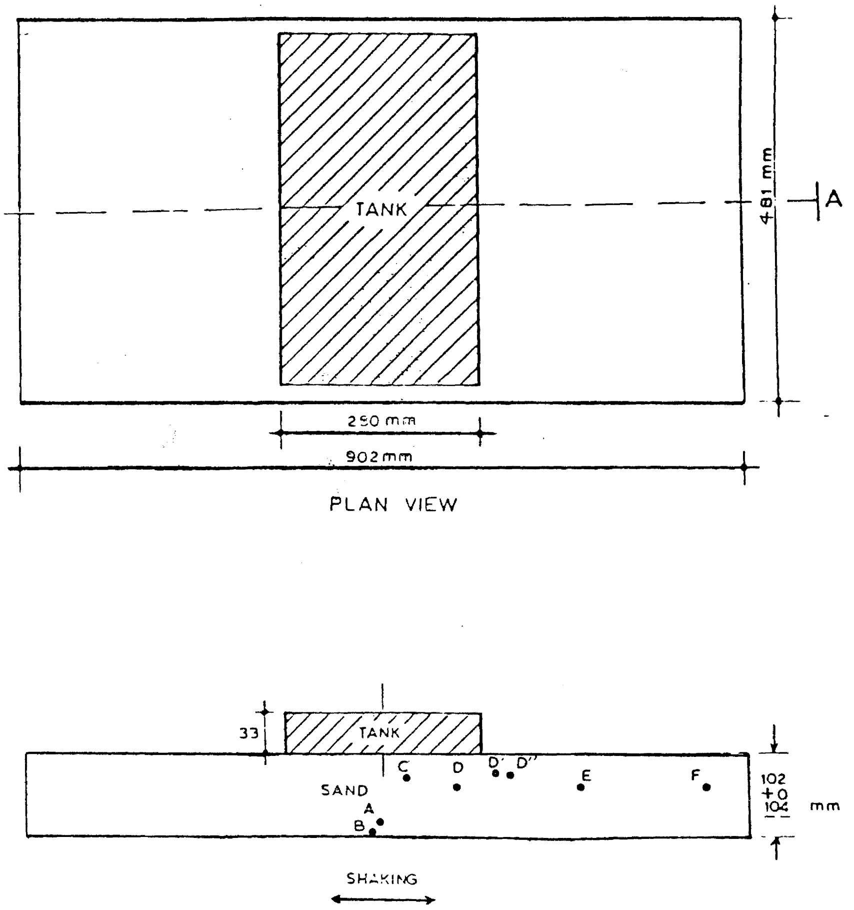

FIG: 17 - GEOMETRY AND LOCATION OF PORE PRESSURE TRANSDUCERS: STRUCTURE FOUNDED UPON SATURATED SAND

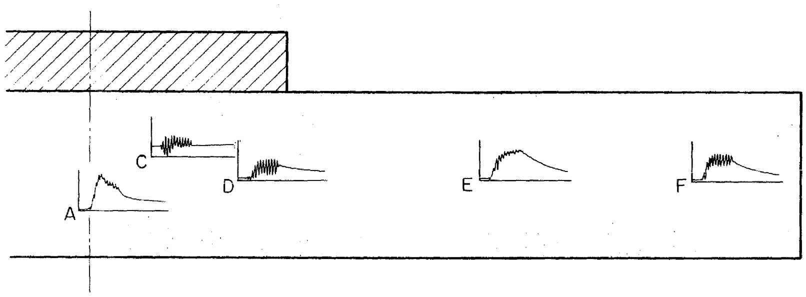

FIG. 18 - EXCESS PORE PRESSURES DURING SHAKING OF MODEL IN FIG. 17.

forces necessary to restrain these strains are calculated, and then the patterns of pore pressure and permanent deformation is found by redistributing these nodal forces throughout the grid. Figure 19 shows excess pore pressures computed after 5 and 10 cycles of shaking, assuming no movement of pore water. Actually some redistribution and dissipation occurred during shaking; however the pattern of the computed pressures - especially the low excess pore pressures under the edge of the structure - is quite similar to that which was observed in the test. Figure 20 gives the computed undrained settlements: the measured settlements of the structure ranged from 0.4 to 0.75 mm, all of which occurred during shaking. Since the experiments were

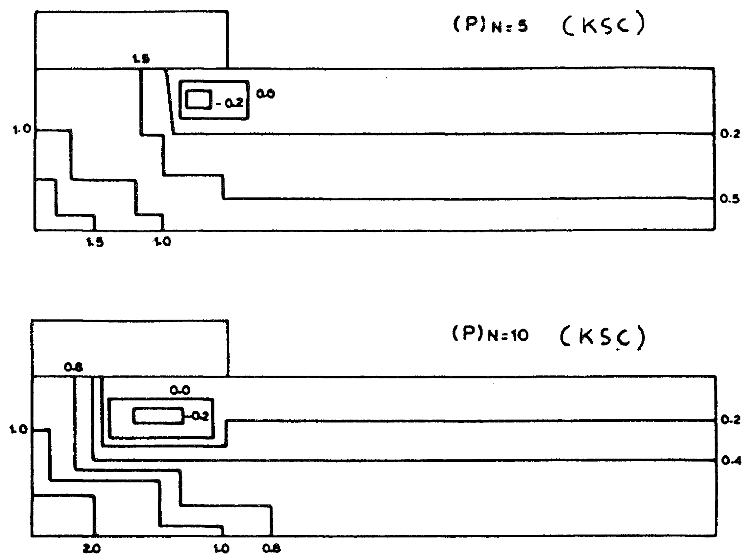

FIG. 19 - COMPUTED EXCESS PORE PRESSURES AFTER 5 AND 10 CYCLES OF SHAKING: MODEL OF FIG. 17.

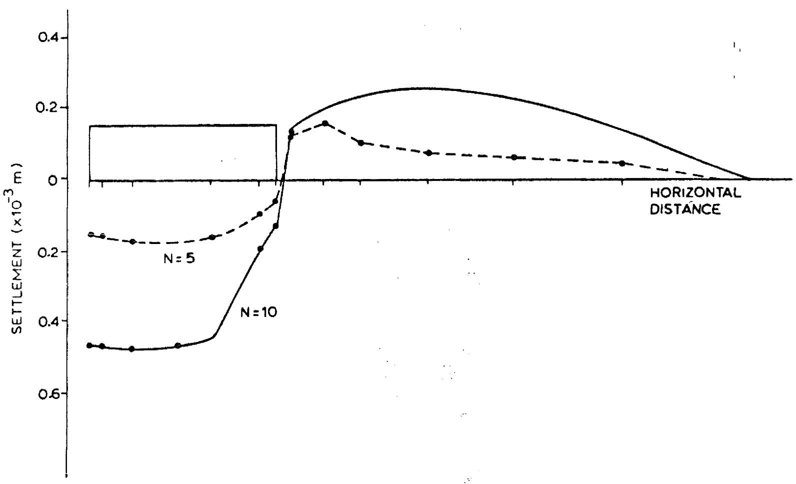

FIG. 20 - COMPUTED UNDRAINED SETTLEMENTS AND HEAVE: MODEL OF FIG. 17.

carried out at a centrifugal acceleration of 80g, the corresponding full-scale settlements would be 3.6 to 6.0 cm.

These are complex problems, and it is demanding of a theoretical model to predict all aspects reasonably well. I believe there is a strong future for combining together theoretical and experimental efforts such as these.

CONCLUDING REMARKS

The main points emphasized in this paper are:

* Earthquake-induced liquefaction flow slides cannot occur unless the residual undrained strength of the sand at large strains is less than the driving shear stress that must be sustained for equilibrium. There is a question as to whether or not this residual strength may, because of departures from truly undrained conditions, differ from undrained steady-state shear strength. Evaluation of these strengths, whether by the standard penetration test (SPT) or undisturbed sampling and testing, still involves considerable uncertainties.

* Analyses which aim at determining "initial liquefaction" with $\sigma \approx 0$ are, strictly speaking, applicable only to level ground. Sites are commonly evaluated using the SPT together with charts correlating blow count to actual experience, although there is a future for the cone penetration test. The "level ground" case is of considerable practical importance. This type of analysis can also be applied to slopes, to identify cases where no slope failure is possible and to indicate situations where there may be excessive permanent deformations.

* Problems involving liquefaction-related permanent deformations or settlements can be quite complex, and there is a great need to validate theories proposed for predicting such deformations. Centrifuge model studies can be very useful in providing data for comparison with theoretical predictions, and several such studies have been summarized in the paper.

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