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The Role of Case Studies in the Evaluation of Soil Liquefaction Potential

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SYNOPSIS Field evidence concerning soil liquefaction is reviewed and a number of case studies are summarized. Examples of the use of case studies in developing an understanding of the liquefaction phenomenon, both on level ground due to earthquake shaking and in slopes due to earthquake and static stress applications are presented.

INTRODUCTION

It is only in the past twenty years or so that studies of soil liquefaction have formed an important part of geotechnical engineering research and practice--though many engineers may well believe that interest and activity in this area since that time has more than made up for previous years of relative neglect. Never-the-less the occurrence of liquefaction-type flow slides can be traced back to ancient times and a general understanding of the phenomenon to the early days of modern soil mechanics.

The earliest reported case of a landslide resulting from soil liquefaction induced by an earthquake may well be that reported by Marinatos (1960)):

> "In the year 373/2 B.C. during a disastrous winter night, a strange thing happend in central Greece. Helice, a great and prosperious town on the north coast of the Peloponnesus, was engulfed by the waves after being levelled by a great earthquake. Not a single soul survived....The next day two thousand men hastened to the spot to bury the dead, but they found none, for the people of Helice had been buried under the ruins and subsequently carried to the bottom of the sea, where they now lie."

Helice was located on deltaic deposits of alluvial sand between the mouths of the Selinus and Cerynites Rivers and about a mile and a half from the coast. However no trace of it now exists, neither on the ground surface nor on the bottom of the sea.

The events leading to the disappearance of Helice and its inhabitants are not immediately clear. A general subsidence of the land area during the earthquake undoubtedly occurred and this alone could have led to flooding of the city. However both Schmidt (1875) and Marinatos, who made detailed studies of the event, concluded that in addition to destruction of buildings by the ground shaking and flooding due to land subsidence, the ground slipped towards the sea possibly as much as half a mile. Marinatos notes that ordinarily it would be expected that building destruction and flooding would lead to some of the dead floating to the surface where they would have been picked up by the Achaeans for burial. It seems reasonable to conclude that only the entrapment of the inhabitants in collapsed buildings, and temporarily liquefied and flowing soils could have led to the recorded facts that no one survived and no dead were found.

Some evidence in support of this concept (Marinatos) is provided by the fact that "the phenomenon was repeated, in exactly the same place though to a lesser degree, during the earthquake of December 26, 1861....Again the soil slipped to the northeast (toward the sea) in the following way: a crack about eight miles long and six feet wide appeared in the earth along the foot of the mountain. A strip of plain 325 ft to 425 ft wide disappeared slowly under the sea along the whole eight-mile length, while the remaining part of the plain sank about six ft and showed many minor cracks and small chasms."

A map of the area showing the extent of cracking in the 1861 earthquake and a drawing of a part of the plain adjacent to the coast, both prepared by Schmidt are extremely revealing. Cracking of the extent indicated must necessarily have been accompanied by lateral translation of the soil, and the presence of sand craters on the drawing is indicative of the probable liquefaction of sand deposits at some depth below the ground surface. In view of the fact that the earthquake of 373 B.C. is estimated to have been about 10 times greater in intensity, the probability that landslides due to sand liquefaction contributed to the disappearance of Helice must be considered extremely high.

Many other landslides due to soil liquefaction induced by earthquake shaking have been reported (Seed, 1968) but a significant number have also occurred in coastal and off-shore deposits due to other causes, generally considered to be tidal waves or fluctuations, rapid erosion or deposition of soil, or construction activities. Similar slides in loose deposits, often involving extensive flow of liquefied soil, have sometimes occurred for no known reason.

Liquefaction of loose sands has also been induced in level ground deposits by earthquake shaking.

A classic example of this type of liquefaction is that which occurred at Niigata, Japan in the Niigata Earthquake of June 16, 1964, but an earlier graphic eye-witness account of this type of phenomenon is that concerning soil behavior in the Ganges plain during the Bihar-Nepal earthquake of 1934 (Geol. Survey of India, 1939):

> "....my car suddenly began to rock in a most dangerous fashion....Owing to the sound of the engine I noticed no noise, but was told such was heard from the west, a deep terrifying rumble. As the rocking ceased, mud huts in the village, on either side of the road, began to fall. To my right a lone dried palm trunk without a top was vigorously shaken, as an irage man might shake his stick, then water spouts, hundreds of them throwing up water and sand were to be observed on the whole face of the country, the sand forming miniature volcanoes, whilst the water spouted out of the craters; some of the spouts were quite six feet high.
>
> "In a few minutes, on both sides of the road, as far as the eye could see, was vast expanse of sand and water, water and sand. The road spouted water and wide openings were to be seen across it ahead of me, then under me, and my car sank, while the water and sand bubbled, and spat, and sucked, till my axles were covered....
>
> "In less than half an hour, I should say, the water spouts ceased to play, though water oozed out of the land and trickled from the mouth of the lesser sand heaps..."

Similar occurrences, often with dramatic consequences have been reported in many other earthquakes.

There can be no doubt that liquefaction is an important cause of soil instability and flow slides and there are numerous case studies to illustrate this statement. Some of the important cases are listed in Tables 1 and 2. In this paper I shall try to demonstrate how case studies have contributed to our present understanding of the phenomenon of liquefaction and of practical methods for evaluating the liquefaction susceptibility of soil deposits.

ORIGIN OF CONCEPT OF LIQUEFACTION

In the modern era of soil mechanics (since about 1915) some of the most dramatic occurrences of flow slides were those associated with failures of hydraulic fill dams. It was in connection with one of these failures, the slide in the Calaveras Dam in California, that the concept of soil liquefaction seems to have been introduced by Hazen into soil engineering terminology. In a classic paper on Hydraulic Fill Dams presented to ASCE in 1920, Hazen wrote as follows:

> "When a granular material has its pores completely filled with water and is under pressure, two conditions may be recognized. In the first or normal case, the whole of the pressure is communicated through the material from particle to particle by the bearings of the edges and points of the particles on each other. The water in the pores is under no pressure that interferes with this bearing. Under such conditions the frictional resistance of the material against sliding on itself may be assumed t be the same, or nearly the same, as it would be if the pores were not filled with water. In the second case, the water in the pores of the material is under pressur The pressure of the water on the particles tends to hold them apart; and part of the pressure is transmitted through the water. To whatever extent this happens the pressure transmitted by the edges and points of the particles is reduced. As water pressure is increased, the pressure on the edges is reduced and the friction resistance of the material becomes less. If the pressure of the water in the pores is great enough to carry all the load it will have the effect of holding the particles apart and of producing a condition that is practically equivalent to that of quicksan
>
> "An extra pressure in the water in the pores of such a material may be produced b a sudden blow or shock which tends to compress the solid material by crushing the edges and points where they bear, or by causing a re-arrangement of particles with smaller voids. An illustration of this ca be seen in the sand on the seashore. Such sand, comparable to dune sand in size, is usually found to be saturated with water for a certain distance above the water level. This condition is maintained by capillarity. If a weight is slowly placed on this saturated sand, there is a slight settlement, the grains of sand coming to firmer bearings, and the weight is carried A sharp blow, as with the foot, however, liquefies a certain volume and makes quick sand. The condition of quicksand lasts fo only a few seconds until the surplus water can find its way out. When this happens the grains again come to solid bearings an stability is restored. During a few secon after the sand is struck, however, it is almost liquid, and is capable of moving or flowing or of transmitting pressure in the same measure as a liquid.
>
> "The thought has occurred to the writer, i looking at the material that slid in the Calaveras Dam, that something of this kind may have happened on a large scale--800,000 cu. yd. of fill flowed for a brief space, and then became solid. It was, in fact, so solid that in examining it afterward, by samples and by borings, it was difficult to see how the material could have flowed--as it certainly did flow.
>
> "It may be that after the first movement there was some readjustment of the material in the toe which resulted in producing temporarily this condition of quick sand, and which destroyed for a moment the stability of the material and facilitated the movement that took place.
>
> "This will not account for the initial mov ment; but the initial movement of some par of the material might result in accumulati

Table 1 Landslides During Earthquakes Due to Soil Liquefaction, 383 BC to 1965

Date	Earthquake	Magnitude	Location	Type of Structure	Soil Type
73 BC	Helice	-	Helice	Coastal delta	-
1755	Lisbon	8.7	Fez	-	-
1783	Calabrian	-	Soriano Laureau Terramuova	River banks River banks River banks	Clays with sand seams Fluvial deposits Fluvial deposits
1811	New Madrid	8	Many	River banks and islands	Fluvial deposits, sands to muds
1869	Cachar		Silchar	River banks	Fluvial sand to clay
1886	Charleston	7.5	Ashley River	River banks	Fluvial and deltaic sands and silts
1897	Assam	8.7	Many	Canal banks	-
1899	Alaska	-	Valdez	Submarine deposit	Deltaic and marine sediments
1901	St. Vincent	-	St. Vincent	Coastal delta	-
1906	San Francisco	8.2	San Francisco area	Hillsides	-
1907	Karatag	8	-	Loess slopes	Loess
1907	Chuyanchinsk	-	-	Loess slopes	Loess
1908	Alaska	-	Valdez	Submarine deposit	Deltaic and marine sediment
1908	-	-	Messina	Submarine deposit	Sand/silt
1911	Alaska	7.0	Valdez	" "	" "
1912	Alaska	7.2	Valdez	" "	" "
1920	Kansu	-	Kansu Province	Loess slopes	Loess
1923	Kwanto	8.2	Tokyo area	Coastal hillsides	-
1925	Santa Barbara	6.3	Santa Barbara	Earth dam	Silty sand
1928	Chile	8.3	El Terriente	Tailings dam	Mining waste
1929	-	-	Grand Banks	Submarine deposit	Sand/silt
1933	Long Beach	6.3	Long Beach	Highway fills	Fills over marshland
1934	Nepal	8.4	Motihari	Lake banks	Alluvium - sand lenses
1935	India	7.6	Quetta	River banks	-
1940	El Centro	7.0	Imperial Valley	Canal banks	Fills on deltaic sands
1941	Garm	-	-	Loess slopes	Loess
1943	Faizabad	-	-	Loess slopes	Loess
1948	Fukui	7.2	Fukui plain	Levees, river banks	Fluvial sands and silts
1949	Chait	7.5	Surchob and Yasman river valleys	Loess slopes	Loess
1950	Imperial Valley	5.4	Calipatria	Canal banks	Deltaic sands
1953	-	-	Suva, Fiji	Submarine deposits	Sand
1954	-	-	Orleansville	Submarine deposits	-
1954	Anchorage	6.7	Rabitt Creek	Embankment	Fill on sand
1957	San Francisco	5.3	Lake Merced	Lake banks	Beach sands
1959	Jaltipan	6.5	Coatzacoalcos	Waterfront fill	Fine sandy silt
1960	Chile	8.4	Rinihue	River banks, coastal fills	Fluvial sands and silts
1964	Alaska	8.3	Valdez Seward	Coastal delta Coastal delta	Silty sands and gravel -
1965	Chile	7.2	Several locations	Tailings dams	Sandy silt and silty sand
1965	Seattle	6.7	Port Orchard Duwamisa	Waterfront fill River terrace	Sand and marine clay Fluvial sands and silts

Table 2 Liquefaction Landslides in Coastal Areas - Not Earthquake Related

	Event	Soil Type	Cause
1.	229 flow slides in Province of Zealand	Fine sand	Seepage forces and erosion associa with large tidal fluctuations. Sli commonly occur at extreme low tide after exceptionally high spring ti
2.	Many slides along banks of Mississippi River, U.S.A.	Sand	Undercutting of river banks
3.	Slides in Trondheim Harbor, 1888	Silty sand	Tidal waves - sliding when wave receded
4.	Slide in Trondheim Harbor, 1930	Silty sand	Not known
5.	Slide in Trondheim Harbor, 1942	Silty sand	Not known
6.	Slide in Trondheim Harbor, 1950	Silty sand	Not known
7.	Slide in Orkdals Fjord, Norway, 1930	Loose sand and soft non-plastic silt	Occurred at exceptionally low tide and preceded by small tidal wave
8.	Slide in Helsinki Harbor, Finland, 1936	Sand	During fill construction
9.	Kitimat, British Columbia, 1974	Fill on clay	Just after low tide
10.	Kitimat, British Columbia, 1975	Fill on clay	No fill being placed; extreme low tide for tidal range of 20 ft
11.	Slide at Howe Sound, B.C., 1955	Silty sand	Extreme low tide
12.	Slide in Folla Fjord, 1952	Sand	Unknown, possibly wave-induced
13.	Rockall (Ancient)	?	Rapid sedimentation
14.	Spanish Sahara (Ancient)	Gravelly clayey sand	Rapid sedimentation
15.	Wahro Bay, Africa (Ancient)	?	Unknown
16.	Copper River, Alaska (Ancient)	Silt/sand	Rapid sedimentation
17.	Wil. Canyon (Ancient)	Silty clay and silt	Rapid sedimentation
18.	Mid Atl. Cont. Slope (Ancient)	Silty clay	Rapid sedimentation
19.	Magdalena River, 1935	?	Rapid sedimentation
20.	Sokkelvik, 1959	Quick clay and sand	Unknown

pressure, first on one point, and then on another, successively, as the early points of concentration were liquefied and in that way a condition comparable to quicksand in a large mass of material may have been produced."

Not only does this excerpt show an excellent understanding of the mechanism of liquefaction but also a deep appreciation of the effective stress principle, which was also being developed at the same time by Terzaghi. For these and other reasons, Hazen must be considered one of the great early workers in the soil mechanics field. It may be noted that, as originally conceived by Hazen, liquefaction is a condition like quicksand which may last for only a few seconds and may be induced either in slopes on level ground such as a beach. This broad cept of liquefaction seems to be generally accepted by most engineers.

It was about 15 years later (1936) that Casagrande wrote his classic paper on the sh strength characteristics of cohesionless soi and introduced the concept of the critical v ratio, followed later by the critical state thereby establishing the principles governin behavior of cohesionless soils under static ing conditions. It was not until the late l however that tests were performed in which liquefaction was induced in test specimens u controlled loading conditions in the laborat (Seed and Lee, 1966; Castro, 1969), and antitative determinations of the stresses using liquefaction or cyclic mobility were le to be made. These tests opened the door to .e possibility of quantitative determinations the liquefaction potential of soil deposits .t practical difficulties associated with the .aracteristics of natural sand deposits pre- .uded their meaningful application, without the .d of empirical rules based on case histories, ıtil recent years. Thus case studies have :cessarily provided the basic guidance for .quefaction potential evaluations, despite :eat advances in the understanding of the basic :inciples controlling soil liquefaction ıenomena.

ꝸACTICAL DIFFICULTIES IN USING LABORATORY ːST DATA FOR EVALUATION OF LIQUEFACTION)TENTIAL OF NATURAL DEPOSITS

ı attempting to use theoretical analyses and ıboratory test data to evaluate the liquefac- ion potential of sand deposits in engineering ractice, a number of difficulties are encoun- ered which relate mainly to accurate char- cterization of the properties of the deposit; hese include:

. The difficulty of obtaining truly undisturbed samples of any sand by even the best undisturbed sampling techniques, unless they involve in-situ soil freezing prior to sampling. In pushing thin wall sampling tubes into unfrozen sands, loose sands are densified to some extent and dense sands are loosened. Thus the properties of the samples used in the laboratory tests may not be representative of those of the in-situ deposit. Furthermore. as illustrated in Figs. 1 and 2, the results of cyclic loading tests to evaluate liquefaction characteristics depend to a large extent on the type and quality of samples used in the testing program. Fig. 1 shows the measured cyclic loading resistance of two sets of samples taken from the same sand deposit, one set by hand-trimming block samples and the other set by good quality "undisturbed sampling" in thin-walled tubes. The results are different by 100% and neither set is likely to reflect the true in-situ properties of the sand (Marcuson and Franklin, 1979). Fig. 2 shows a comparison of the known cyclic loading resistance of a large block of dense sand and the measured resistance of high-quality undisturbed samples taken in thin-wall tubes from the same block. In this case the measured cyclic loading resistance of the "undisturbed samples" was only about 30% of that of the sand block from which they were extracted (Seed et al., 1982). The effects of sampling disturbance on the cyclic loading resistance of medium dense sands is likely to be much smaller than the values indicated by the data in Figs. 1 and 2, but because of the great difficulties in obtaining and testing truly undisturbed samples of sand, considerable judgment may be involved in the interpretation of laboratory test data.

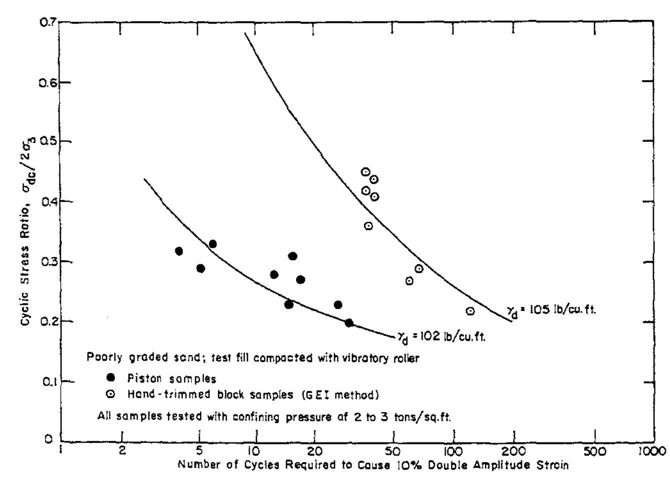

Fig. 1 Influence of Method of Sampling on Cyclic Loading Resistance of Dense Sands

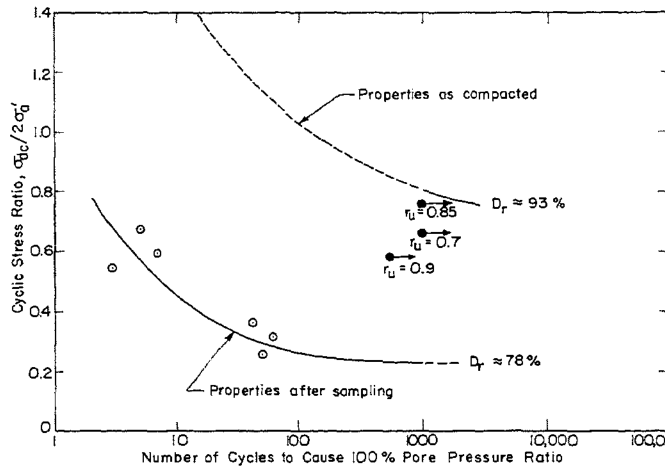

Fig. 2 Effect of Sample Disturbance on Cyclic Loading Resistance of Dense Sand

2. The difficulty in selecting representative samples for use in a test program, invariably limited in the number of samples which can practically be tested, from a deposit of considerable non-uniformity. The non-uniformity of sand deposits has long been recognized (Terzaghi and Peck, 1949) and unless great care is exercised, extraction of samples from one or two locations may not in any way provide representative conditions for use in a test program.

3. Difficulties in simulating field loading conditions in laboratory tests. For example the field loading conditions for a soil element subjected to earthquake shaking involve multidirectional shear in the horizontal plane coupled with simultaneous vertical stresses. It is virtually impossible to simulate these effects under controlled conditions in the laboratory and thus laboratory tests are necessarily idealized approximations

(often made useful by calibrations with field performance) of actual field conditions.

4. The difficulty in establishing the drainage conditions for field deposits. In the more elaborate analyses of soil liquefaction during earthquakes, the effects of simultaneous generation and dissipation of pore water pressures in soil deposits are computed, based on some concept of the boundary drainage conditions in the field; yet the existence of a thin layer of relatively impervious soil within an otherwise permeable deposit could totally change the pore pressure dissipation characteristics.

For these and other reasons, it seems unlikely that the behavior of natural deposits in the field can be computed simply by analysis and laboratory testing and that the usefulness of such approaches must always be calibrated by comparison of analytical results with field performance established by case histories. Typical examples of this are presented in the following section.

EXAMPLES OF THE USE OF CASE STUDIES TO EVALUATE LIQUEFACTION

1. Level Ground Liquefaction Due to Earthquake Shaking

The development of a quicksand-like condition (liquefaction) on level ground during and following earthquake shaking has frequently been observed and over the past twenty years considerable effort has been devoted to predicting the conditions under which this may occur.

Because of the difficulties in sampling and testing undisturbed samples of sand noted above, it has been found practically more expedient and reliable to develop a procedure for evaluating the liquefaction resistance of sands through the use of case histories in which the field behavior of sands is correlated with a suitable index of liquefaction resistance such as the results of the Standard Penetration Test.

In using this approach, earthquake shaking of in-situ deposits is used as the test excitation mechanism, a perfect loading condition, and the field behavior, measured in terms of the cyclic stress ratio induced by the earthquake, is correlated directly with the penetration resistance, N_1, of the soil. The cyclic stress ratio developed under field loading conditions can readily be computed from the equation:

$$\frac{(\tau_h)_{av}}{\sigma_o'} \simeq 0.65 \, \frac{a_{max}}{g} \cdot \frac{\sigma_o}{\sigma_o'} \cdot r_d$$

where a_{max} = maximum acceleration at the ground surface

σ_o = total overburden pressure on sand layer under consideration

σ_o' = effective overburden pressure on sand layer under consideration

r_d = a stress reduction factor varying from a value of 1 at the ground surface to a value of 0.9 at a depth of about 30 ft.

Values of this parameter have been correlated for sites which have and have not liquefied during actual earthquakes, with the standard penetration resistance of the sands underlying the sites, expressed by the normalized penetration resistance N_1 of the sand deposit involved (Seed et al., 1983). In this form of presentation N_1 is the measured penetration resistance corrected to an effective overburden pressure of 1 ton/sq. ft. or 1 ksc and can be determined from the relationship

$$N_1 = C_N \cdot N$$

where C_N is a function of the effective overburden pressures at the depth where the penetration test was conducted. Values of C_N can be determined from the chart in Fig. 3, which is based on studies conducted at the Waterways Experiment Station (Bieganousky and Marcuson, 1976; Marcuson and Bieganousky, 1976).

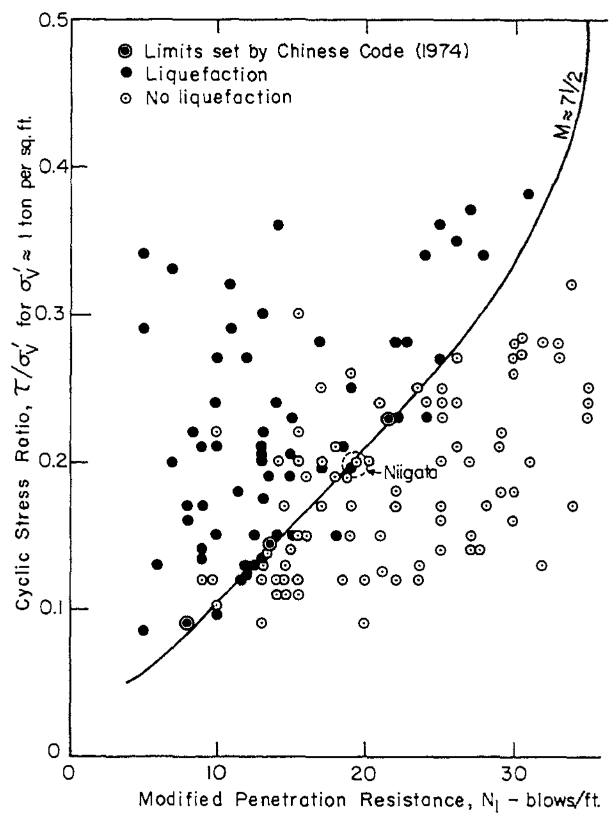

Fig. 3 Correlation Between Field Liquefaction Behavior of Sands Under Level Ground Conditions and Standard Penetration Resistance

The results of over 130 individual studies shown in Fig. 4 from which it may be seen that the possibility of liquefaction occurring can be determined with a good degree of assurance from the data presented. The line on the chart is a lower bound line and sites plotted below

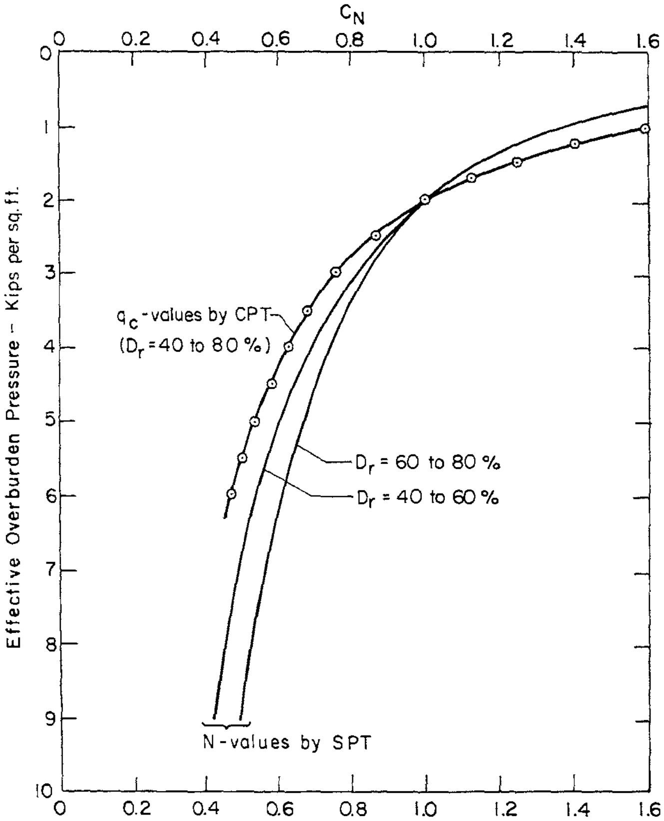

Fig. 4 Relationships Between C_N and Effective Overburden Pressure

line are not likely to show evidence of liquefaction in any earthquake of magnitude 7-1/2 or less. The data points shown in Fig. 4 are from site studies in the United States, Japan, China, Guatemala and Argentina and thus represent a wide range of geographical locations and conditions. The extent of this field data, based on case histories, makes the evaluation of liquefaction potential by this approach a more reliable procedure than one involving the uncertainties associated with sampling and laboratory testing of sands in most cases.

2. Earthquake-Induced Liquefaction in Earth Dams

Evaluating the seismic stability of earth dams against the possibility of slope failures due to soil liquefaction is a considerably more complex problem then the evaluation of level ground liquefaction since it involves determining not only the zones of the embankment where liquefaction (as produced by high residual pore pressures and loss of strength) might occur, but also the residual strength of the "liquefied soil." Furthermore these evaluations must be made for elements of soil in the dam having widely different initial (pre-earthquake) stress conditions and different magnitudes of superimposed earthquake stresses. It is unlikely that any credible method of evaluating seismic stability under these complex conditions could be developed without the aid of case histories to calibrate the method so that it provides results in accordance with known field performance.

In fact much of what we know about dam performance during earthquakes comes from case studies, including the possibility of instability due to soil liquefaction. Prior to 1971 there was apparently a general belief among earth dam engineers that failure of a major dam, even a hydraulic-fill dam, due to earthquake shaking was not a likely occurrence and that seismic design studies were not an essential component of safety evaluations. This concept was dispelled by the major slope failure in the Lower San Fernando Dam in 1971 (Seed et al., 1975). As a matter of fact a number of dams and tailings dams have suffered liquefaction type slides due to earthquake shaking in the past 60 years including the Sheffield Dam, 1925, Baharona Tailings Dam, 1928, El Cobre Tailings Dam, 1959, Lower San Fernando Dam, 1971, Mochi-Koshi Tailings Dam, 1978 and careful attention is now given to this aspect of dam design.

Experience with many dams subjected to earthquake shaking shows that when they are constructed of materials which do not lose any significant strength as a result of the earthquake shaking (as is the case when liquefaction occurs), they suffer only minor deformations even under very strong shaking conditions (Seed et al., 1978). Thus it is only for embankments constructed on or of loose to medium dense cohesionless soils, in which some degree of liquefaction may occur, that major stability problems are likely to develop.

In cases where liquefaction is the cause of embankment instability, the loss of strength may occur either during or following the earthquake shaking. When it occurs during the earthquake shaking it is a direct result of the pore pressure build-up by the cyclic stress applications but when a liquefaction failure occurs after the earthquake shaking, it may be due to a progressive build-up of pore pressure with time triggered by the cyclic stress applications or it may be due to a redistribution of pore water pressure within the embankment. In either case the earthquake-induced stresses are necessarily the trigger mechanism producing a loss of shear resistance in the soil, and sliding occurs when the shear resistance of the soils drops to a level at which it is equal to the shear stresses in the embankment due to gravity effects and possibly some inertia effects. Thus the overall problem for the design engineer involves three parts:

1. Determining the level of earthquake shaking required to trigger any degree of loss of strength or soil liquefaction in the embankment.

2. Determining the extent of the zone of soil liquefaction which may develop if the triggering shaking level is exceeded.

and 3. Determining whether the combined resistance of any non-liquefied zones and the residual strength of the liquefied zones is sufficient to prevent a major slide, bearing in mind that the residual strength of liquefied sand may decrease progressively to a steady-state value with increasing strain in the early stages of deformation.

All of the design problems listed above are illustrated by case studies. Thus for example, a potentially liquefiable sand clearly existed in the Lower San Fernando Dam (since it eventually liquefied in the earthquake of 1971). However the same sand had previously been subjected to lower levels of earthquake shaking on numerous occasions since it was first constructed in 1915, with no detrimental effects. The same type of behavior is illustrated by the behavior of the sand deposits underlying the city of Niigata, Japan. This sand was also shaken by numerous earthquakes over a 350 year period but it did not undergo the extensive liquefaction that occurred during the stronger 1964 Niigata earthquake (Fig. 5). In effect this means that a potentially liquefiable embankment or deposit may be perfectly safe if it exists in an area of low seismic activity but it may be hazardous if it exists in an area of high seismic activity. Determining the level of shaking which will trigger liquefaction is an essential component of the seismic slope stability evaluation problem, as it is for level ground liquefaction problems.

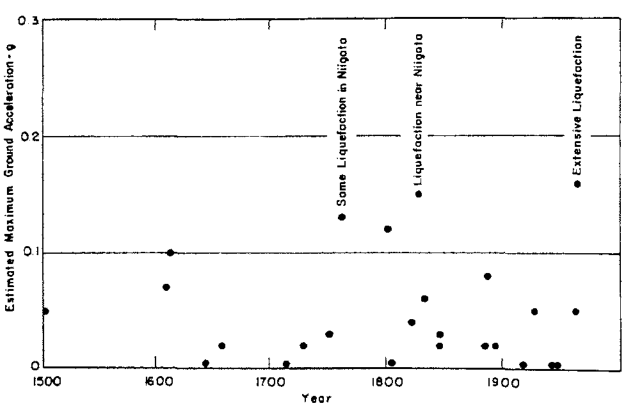

Fig. 5 Estimated Peak Ground Accelerations Developed by Earthquakes in Niigata, Japan

Similarly it is necessary to be able to predict the extent of the zone of liquefaction which may develop within an embankment if a meaningful evaluation of seismic stability is to be made. This is illustrated by the case study of the Upper San Fernando Dam in the San Fernando earthquake of 1971 (see section in Fig. 6). It seems likely that liquefaction occurred within the embankment as a result of the 1971 San Fernando earthquake (Seed et al., 1973), but not over such an extensive zone as in the Lower San Fernando Dam. No extensive sliding occurred, presumably because the combined strengths of the soil in the non-liquefied zone and the residual strength of the liquefied soil were sufficient to withstand any shear stresses induced by gravity effects and earthquake shaking effects.

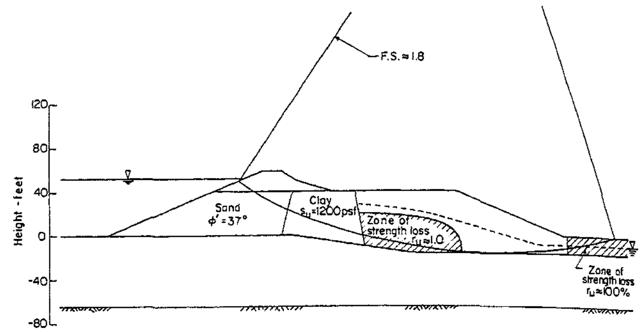

Fig. 6 Probable Condition of Upper San Fernando Dam just after 1971 San Fernando Earthquake

Clearly in such cases it is important to be able to evaluate both the resistance to sliding of non-liquefied zones and the residual strength of any liquefied zone or zone where high residual pore pressures are developed. The development of a limited zone of liquefaction in the central part of an embankment is not usually a source of instability because of the resistance provided by the non-liquefied soil. Sometimes the residual strength of the "liquefied" soil may also be large enough to prevent a flow slide from occurring.

Case studies can provide a means for evaluating both the extent of the zone of liquefaction within a dam and the residual strength of the liquefied soil. Thus for example, in the case of the Sheffield Dam, failure occurred due to downstream sliding of the entire embankment as a result of liquefaction occurring under essentially the entire base; in effect the embankment was pushed downstream by the water pressure acting on the upstream face (Seed et al., 1969). A simple calculation shows that if liquefaction occurred all all along the base, the residual strength of the liquefied soil when sliding occurred would be about 50 psf.

Field studies of the Lower San Fernando Dam indicated 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 would be as shown in Fig. 7. Since sliding occurred 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 was about 800 psf.

Guided by the results of such case studies concerning the mechanism by which liquefaction-induced slides occur, the problem for the design engineer is to develop procedures for predicting the key features of embankment performance. Procedures for exploring whether a given level of earthquake shaking will induce liquefaction or

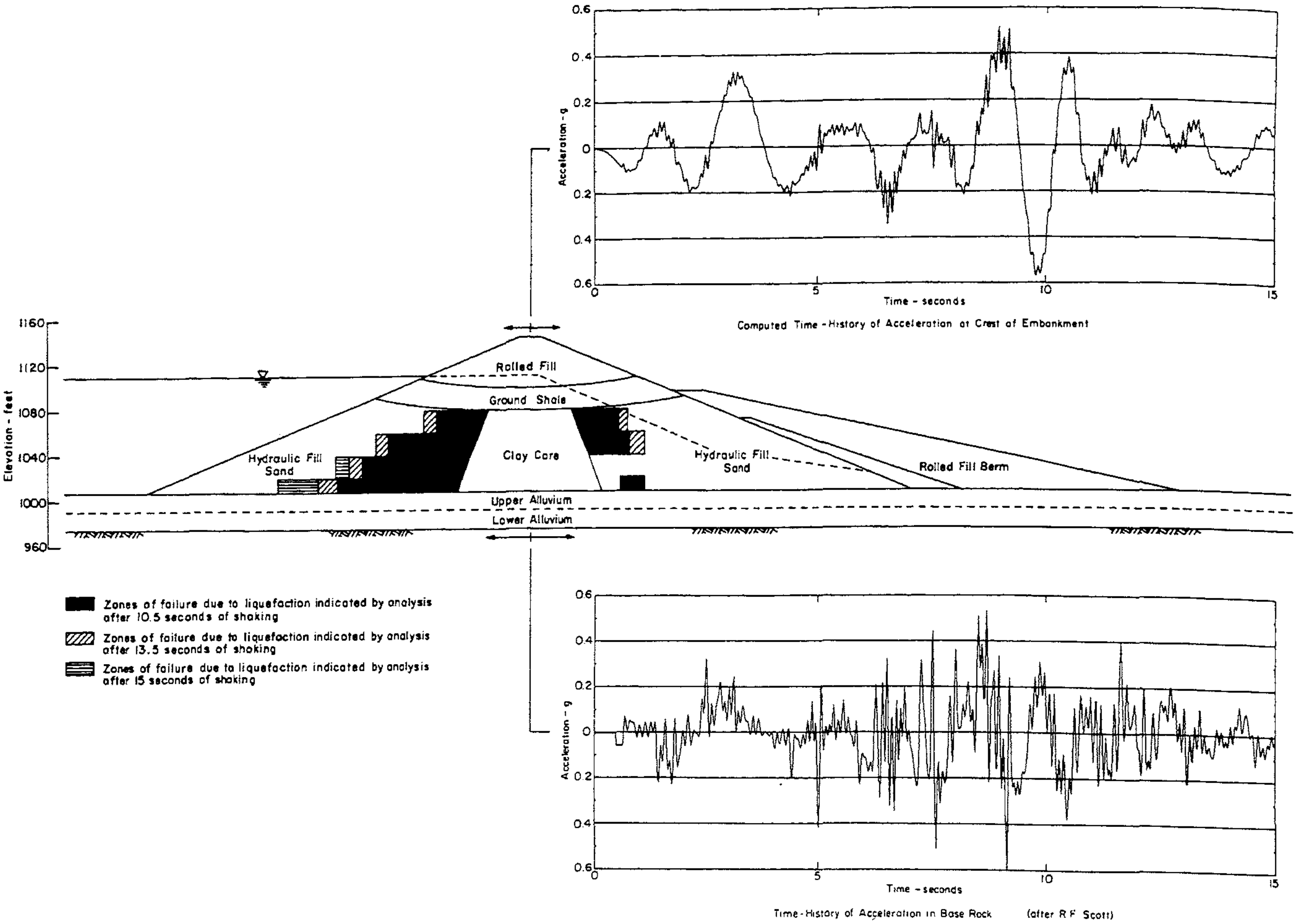

Fig. 8 Predicted Zone of Liquefaction for Lower San Fernando Dam

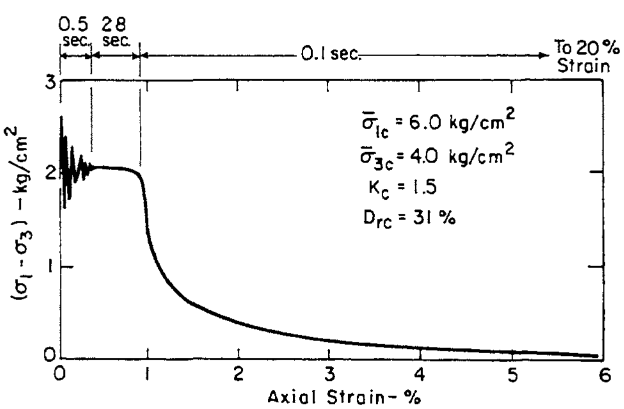

Fig. 9 Delayed Liquefaction Triggered by Cyclic Loading (after Castro, 1978)

The nature of the triggering mechanism which causes liquefaction under these conditions seems to warrant more attention then it has received, in the hope that an improved understanding of numerous currently unexplainable liquefaction-type failures can be developed.

CONCLUSION

The case studies described in the preceding pages were selected to show how careful analyses of such events has furthered the general understanding of the phenomenon of soil liquefaction. In fact, case studies have been the primary means of learning about this phenomenon in the field, and laboratory and experimental studies have served primarily as a means for developing an improved understanding of the phenomena observed. It is clear that we are still learning from case studies, even though we are approaching the point of understanding certain aspects of the problem such as the mechanism of level ground liquefaction induced by earthquakes. Nevertheless there is still a lot to learn about the development of liquefaction failures in natural slopes. A wledge of case histories is likely to be the inant method of evaluating such problems in ineering practice for some years to come.

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