# SOIL AND FOUNDATION BEHAVIOUR DURING EARTHQUAKES

by

H. Bolton Seed*

## I. Earthquake Effects on Soils and Foundations

One of the main tasks of the engineer in producing adequate design is to clearly recognize the problems involved and the manner in which failures might occur. Thus it is important to recognize the kinds of soil problems that can develop during earthquakes, especially those influencing the foundations and performance of structures.

It has long been recognized that vibration is an effective means of compacting cohesionless soils. Thus it is not surprising to find that the ground vibrations caused by earthquakes often lead to compaction of cohesionless soils and associated settlement of the ground surface. If the ground is near the coast line, then extensive land areas may be submerged. Fig. 1 shows an island in Chile[1] which was partially submerged during the Chilean earthquake of 1961. Similar effects occurred in the Alaskan earthquake of 1964[2]. At Portage, for example, the underlying rock dropped some three or four feet, but the ground surface dropped some seven or eight feet. Thus there was, in fact, four or five feet of settlement of the ground surface caused by compaction of the cohesionless sediments overlying the rock. As a result, the town was under water at high tide and residents had to be evacuated.

Settlements of this type may cause differential movement of engineering structures. Fig. 2 shows a bridge in Portage. The piers and abutments were supported on piles, but the abutment piles extended to a greater depth than those under the piers. As a result, the piers settled more than the abutments and the deck now spans across the stream bed, unsupported by the piers on which it is supposed to be resting.

If the sediments which compact during an earthquake are saturated, then water from the voids is forced to the ground surface where it emerges in the form of mud spouts or sand boils. Fig. 3 shows a series of sand boils which developed during the recent earthquake in Nigata, Japan[3]. In many cases an entire area has been covered with boils of this type.

Often the water rushes from the ground surface with considerable velocity. Reports from Portage, for example, speak of water spouting 100 feet into the air.

---

*Professor of Civil Engineering and Chairman, Department of Civil Engineering, University of California, Berkeley. Member of Executive Committee, Soil Mechanics and Foundation Division, ASCE and Department of Soils Geology and Foundations, Highway Research Board.

The upward flow of water resulting from compaction of saturated deposits will often turn the soil into a "quick" condition. It may be recalled that quicksand is caused by an upward flow of water through the soil. If the soil becomes quick or liquefies, it loses its supporting capacity with the result that structures or firmer areas may settle into the ground. Fig. 4 shows an area that liquefied during the Chilean earthquake. The hummocky topography resulted from the even settlement, and the building settled considerably into the ground. Settlement of a bridge abutment in Nigata is shown in Fig. 5. Perhaps the most dramatic case of foundation failure due to liquefaction that has been recorded to date is the severe tilting of blocks of apartments in Nigata as a result of the recent Japanese earthquake. One apartment building tilted through an angle of about 80°.

If a liquefied soil mass is not confined on one side, then it flows towards that side producing a mass movement of soil termed a flow-slide. Fig. 6 shows a flow-slide which occurred in Kansu Province, China, during an earthquake in 1920.[4] The soil involved was a loose deposit. Large flow-slides occurred over an area 100 square miles in extent. In one area a piece of roadway, with its border of poplar trees, was transported and set down one mile from its original location. Some reports say that 200,000 people were killed during these enormous land movements.

Flow-slides also occurred in Valdez and Seward during the recent Alaskan earthquake. At Seward much of the slide was underwater, but a substantial portion of the dock area was carried away by the slide. Flow-slides occurred around Lake Merced during a very small earthquake near San Francisco in 1957 and sections of the highway were carried some distance into the lake. Large lateral translations of soil masses are characteristics of flow-slides.

Very often loose saturated sands occur in thin seams or layers underlying firmer materials. In such cases liquefaction of the sand may cause a large mass of land to slide laterally along its base (see Fig. 7). When this happens, a zone of soil at the back end of the sliding mass sinks into the vacant space, forming a depressed zone known as a "graben". This type if movement occurred extensively in Anchorage during the Alaskan earthquake. It can, of course, cause extensive damage to buildings. Structures on the main slide mass are translated laterally, often without significant damage, but buildings in the graben area are subjected to such large differential settlements that they are often completely destroyed. Furthermore, buildings near the toe of the slide area where the soil pushes outwards are often heaved upwards or pushed over by the lateral thrust.

Fig. 8 shows an aerial view of the L-Street slide area in Anchorage.[5] The graben can readily be discerned, winding its way across the landscape. In this slide, the land mass moved about 15 feet, resulting in the formation of a graben about 10 feet deep and 100 feet wide.

The disastrous effect on a building straddling the graben is shown in Fig. 9, while Fig. 10 shows the effect on buildings at the toe of the slide. The large building was heaved upwards in the middle and sags on both sides. A similar type of slide occurred along 4th Avenue in Anchorage, and another, with rather more extensive land movement, in the Government Hill area. This latter slide wrecked a school and cut some buildings completely in two.

Finally, in connection with liquefaction, it should be recognized that fills placed behind quay walls and bulkheads are often in a loose condition. Being adjacent to the water, they are also saturated. Thus, they are very vulnerable to liquefaction during major earthquakes. The liquefied soil will often push out the wall or flow around the sides of the wall. Extensive failures of this type occurred in the Chilean earthquake of 1960 and the Nigata earthquake of 1964.

Major damage can also result from failure of clay soils. Such a slide occurred in the soft clay soils underlying the Turnagain area of Anchorage during the Alaskan earthquake.[5] The slide area (see Fog. 11) was about 2 miles in length along the coast line and extended inland an average of 900 feet.

In addition to those problems already described, damage may also result from failure of retaining walls and bridge abutments to withstand the extra soil pressure exerted on them during an earthquake. Fig. 12 shows a bridge abutment in Chile which was pushed out by the soil behind it, causing a change in configuration of the bridge. Less sturdy bridge structures may collapse completely, as was the case for many bridges in the Portage area of Alaska (see Fig. 13).

## II. Soil Behaviour Under Earthquake Loading Conditions

In order to determine the possibility of damage resulting from earthquake effects on soils it is necessary to evaluate the characteristics of the deposits in some appropriate way. The susceptibility of a sand to liquifaction is controlled to a large extent by its relative density and this characteristic can be conveniently evaluated in the field by means of the standard penetration test. Fortunately the experience of Nigata[3] has provided valuable information on the characteristics of the sand deposit in the area in which major danger occurred. The soil conditions in this area consisted of a loose sand (standard penetration resistance $N \approx 7$) to a depth of about 25 feet after which the density increased relatively uniformly to a penetration resistance of $N = 30$ at a depth of about 50 feet. The ground water table was about 3 feet below the ground surface. Case histories of this type are invaluable in extending engineering experience to other areas.

In cases where penetration tests cannot readily be conducted the liquefaction potential of a sand can be studies by laboratory tests in which samples representative of in-place conditions are subjected to cyclic loading. During an earthquake an element of soil in the ground is subjected to a complex system of deformations resulting from the erratic sequence of ground motions induced by the earthquake. However, in many earthquakes a major part of soil deformations may be attributed to the upward propagation of shear waves from underlying layers so that an element of soil may be considered to be subjected to a series of cyclic shear strains which reverse directions many times during the earthquake. Before the earthquake there is no shear stress on the horizontal planes. During the earthquake the normal stress on these planes remains constant while cyclic shear stresses are induced for the duration of ground shaking.

Such deformation conditions can best be reproduced in the laboratory by a simple shear test conducted under cyclic loading conditions. However, they may also be reproduced with reasonable accuracy for saturated soils by cyclic loading triaxial tests in which a sample is initially consolidated under an ambient pressure condition and then subjected to cyclic deviator stress applications.[6]

The results of a test of this type performed on a sample of loose sand are shown in Fig. 14. In this test a cyclic deviator stress, $\alpha_{dp}$ of constant amplitude (±1.5 kg per sq cm) was applied with a frequency of 2 cycles per second to a sample of saturated sand under a confining pressure of 5 kg per sq cm and the resulting changes in axial strain and pore-water pressure were recorded. The sand was sufficiently pervious that pore-pressure equalization in the sample occurred almost immediately after the stress applications. The test data in Fig. 14 show the changes in stress, strain and pore-water pressure with time.

It will be seen that during the first 60 cycles of stress application the sample showed no noticeable deformation although the pore-water pressure built up gradually. However, during the 61st stress cycle the pore pressure suddenly increased to a value equal to the externally applied confining pressure and the sample developed large strains which in the 63rd cycle exceeded 20 percent; in fact the soil had liquefied, the effective confining pressure was reduced to zero and over a wide range of strains the soil could be observed to be in a fluid condition.

Of particular interest is the suddenness of the liquefaction phenomenon; for eight stress cycles the soil behaved as a solid material yet in the course of one more cycle it was transformed into a fluid condition. This type of behaviour is characteristic of loose sands. In some cases similar samples have withstood several hundred stress cycles without noticeable deformation but have then liquefied suddenly in the course of one or two additional cycles.

It should perhaps be noted that for given conditions of sample density and confining pressure, the number of stress cycles required to cause liquefaction decreases as the magnitude of the cyclic deviator stress is increased. For the same test conditions as those for the sample shown in Fig. 14 the relationship between cyclic stress and number of cycles to cause failure is shown in Fig. 15. The form of this relationship is typical of those for other test conditions.

It is worthy of note that the stresses required to cause liquefaction of saturated sand in cyclic loading tests are much smaller than those required to cause failure under static loading conditions. In fact, soil behaviour under cyclic loading is quite different from that developed under static loading conditions and it is impossible, at the present state of knowledge, to predict the behaviour under cyclic loading conditions from the results of tests performed under conventional static loading conditions.

It should also be noted that the stress conditions to which a soil element is subjected during an earthquake are quite different from those involved in a transient loading test used for investigating other dynamic effects on soils. In a transient loading test an initially unstressed sample of soil is loaded to failure in a very short period of time, as illustrated in Fig. 16a. Under earthquake loading conditions, an initially stressed soil element is subjected to a series of stress pulses, none of which would necessarily cause failure by itself, but the cumulative effect of which is to induce failure or significant deformations. This type of loading is illustrated in Fig. 16b, where for convenience the stress pulses are shown to be of equal magnitude and may be considered to represent only the major pulses developed during an earthquake.

This difference is illustrated by the data presented in Fig. 17. Samples of soft clay were first tested under normal rate of loading conditions and transient loading conditions. The normal compressive strength was found to be 0.4 kg per sq cm and the transient strength for a time of loading of about 0.1 second, was 0.77 kg per sq cm, almost 100 percent larger. A third sample was then loaded two-thirds of the way towards failure in a normal loading test and allowed to remain under this load until deformations had stopped. At this stage it was subjected to a single transient stress pulse (time of loading about 0.1 second) which increased the stress to a value slightly more than the normal strength of the soil, but markedly below the transient strength of the soil. The only effect of this stress pulse was to increase the axial strain of the sample by about 0.2 percent, as shown in Fig. 17a. Finally, the same stress pulse was repeated 20 times; the deformation increased progressively with each application, and on the 21st application the specimen failed completely as shown in Fig. 17b. The maximum stress applied to the specimen was never more than 56 percent of the transient strength but the series of 20 transient pulses was able to induce failure.

It is apparent that deformations under such loading conditions are markedly different from those occurring under transient loading conditions.

A comprehensive test program to study the behaviour of clays under cyclic loading conditions has been conducted at the University of

California.[7] The procedure in a typical test is illustrated by the data in Figs. 18 and 19 for a compacted silty clay having a degree of saturation of about 95 percent. A series of samples of the clay soil to be tested were prepared and the strength of the soil was determined by a conventional undrained loading procedure; the stress-strain relationship for the soil determined in this way is shown by the dashed line in Fig. 7 and the strength by the dashed line in the upper part of Fig. 6. A second sample of the soil was then loaded in the same way to a stress level expressed as some proportion of the normal undrained strength (67 percent for the sample in Figs. 18 and 19) and allowed to come to equilibrium. For the data shown in Fig. 18, this condition was achieved in about 30 minutes. At this stage a series of 100 transient stress pulses were applied and the resulting deformations of the sample were recorded. The progress of deformation with time is shown in Fig. 18 and the stress vs strain relationship for the sample is presented in Fig. 19. Following the stress pulsations, the original sustained stress was left on the sample for a period of time until deformations ceased and the sample was then loaded to failure in the normal fashion.

For the test illustrated in Figs. 18 and 19, the sample deformed almost 5 percent under the initial sustained stress; however the superposition of 100 transient stress pulses caused an additional axial strain of almost 11 percent, even though the maximum stress applied throughout the loading operation did not exceed 90 percent of the normal strength (see Fig. 19). The combination of stresses used in this test (sustained stress equal to 67 percent of the normal undrained strength and pulsating stress equal to 23 percent of the normal strength) did not produce failure but they did induce a deformation of the sample sufficient to cause severe distortion.

By reading the strain after different numbers of transient stress applications the deforming effects of different combinations of sustained and pulsating stresses can readily be determined.

It may be seen from Fig. 19 that the magnitude of the deformations occurring in such a test (assuming the frequency and duration of the stress pulses are maintained constant) will depend, for any given soil, on three factors:

(1) the magnitude of the sustained stress
(2) the magnitude of the superimposed pulsating stress
and (3) the number of stress pulses applied.
Accordingly, test programs have been developed to investigate the effects of different combinations of these three factors.

A typical example of the relationship between sustained stress, cyclic stress and number of stress cycles causing failure of a silty clay soil as shown in Fig. 20. It has been found that the form of the relationship depends on the nature of the loading conditions (onedirectional or two-directional), the soil type, the frequency and duration of the pulsating stresses, the number of stress pulses and the form of the stress pulse although the latter does not appear to be too significant within a reasonable range of variations in the practical range of stress conditions. However, the form of the stress pulse can have a marked effect on the number of stress pulses required to cause failure at certain stress levels.

Test programs of the type described above can throw considerable light on soil behaviour under earthquake loading conditions. A method of utilizing the information for the design of embankments is described in Reference (8) and similar approaches have been used for determining soil parameters for evaluating the response of bridges on long piles passing through deep deposits of soft clay.[9] Analvses of this type are likely to find increasing use in foundation engineering practice.

## References


1. Rosenbluth, E., "Chilean Earthquakes of May 1960 -- Their Effects on Civil Engineering Structures", Reviska Ingenieria, January 1961.

2. U.S. Department of the Interior, "Alaska's Good Friday Earthquake, March 27, 1964", Geological Survey Circular 491: 1964.

3. Japan National Committee on Earthquake Engineering, "Nigata Earthquake of 1964", Proceedings Third World Conference on Earthquake Engineering, February, 1965.

4. Close, V. and E. McCormick, "Where the Mountain Walked", National Geographic Magazine, May, 1922.

5. Shannon and Wilson, Inc., "Anchorage Area Soil Studies, Alaska", Report to U.S. Army District, Anchorage, Alaska, August, 1964.

6. Seed, H.B. and K.L. Lee, "Studies of the Liquefaction of Sands Under Cyclic Loading Conditions", Soil Mechanics and Bituminous Laboratory, University of California, Berkeley, 1965.

7. Seed, H.B. and Clarence K. Chan, "Strength of Clays Under Simulated Earthquake Loading Conditions", Soil Mechanics and Bituminous Laboratory, University of California, Berkeley, 1965.

8. Seed, H.B., "A Method for Earthquake Resistant Design of Earth Dams", Soil Mechanics and Bituminous Laboratory, University of California, Berkeley, 1965.

9. Parmelee, R.A.; J. Penzien; C.F. Scheffey; H.B. Seed; G.R. Thiers, "Seismic Effects on Structures Supported on Piles Extending Through Deep Sensitive Clays", Report No. SESM 64-2, Institute of Engineering Research, University of California, Berkeley.

Fig. 1

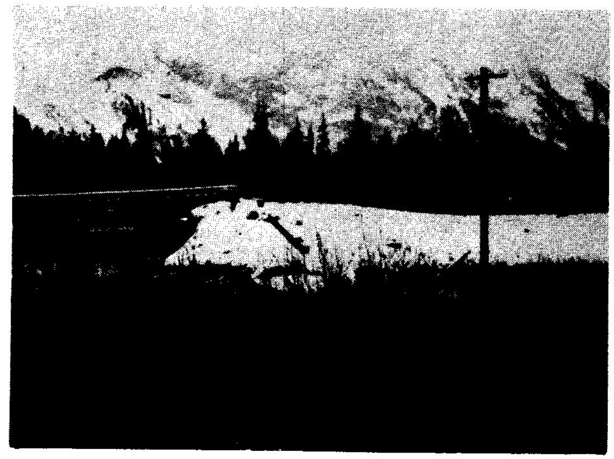


Fig. 2

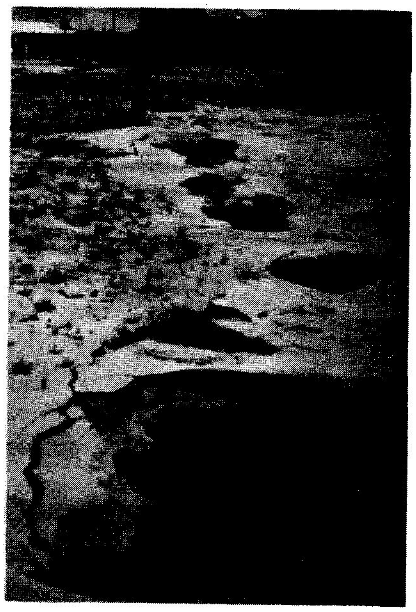


Fig. 3

Fig. 4

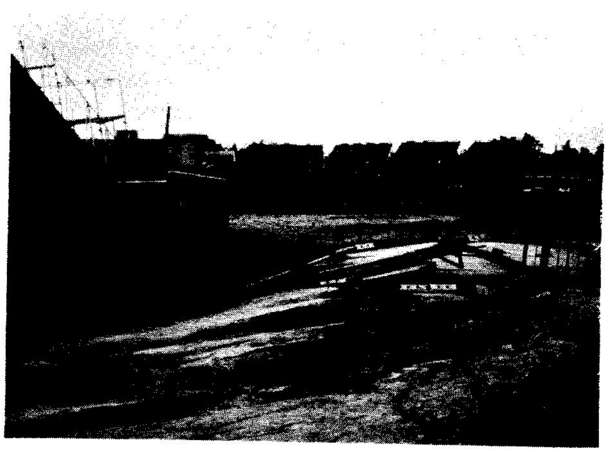


Fig. 5

Fig. 6

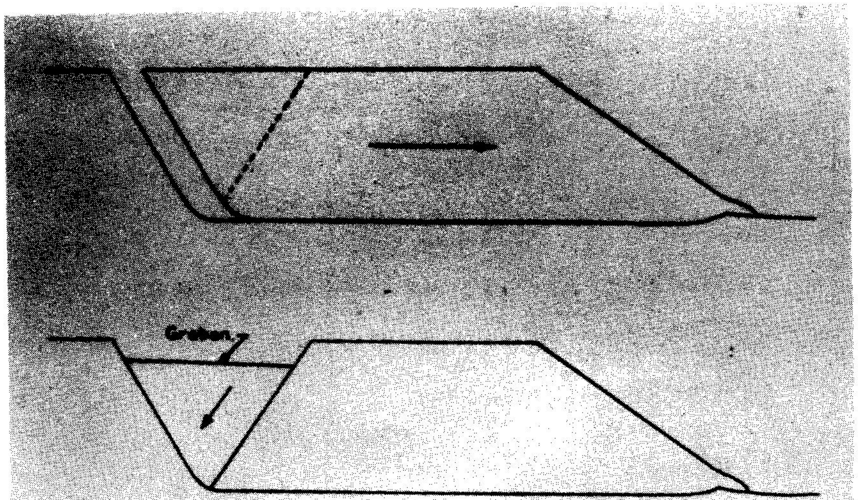



Fig. 7

Fig. 8

Fig. 9

Fig. 10

Fig. 11

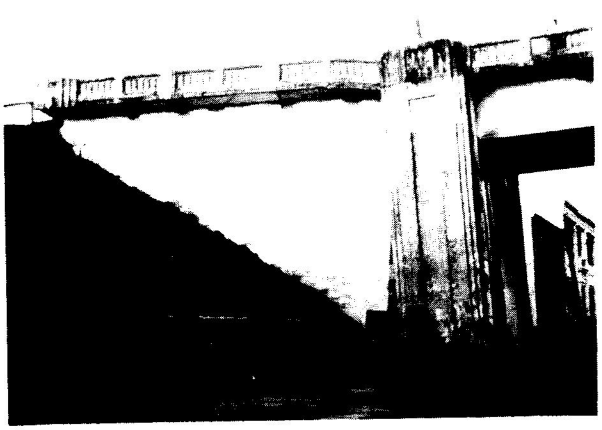


Fig. 12

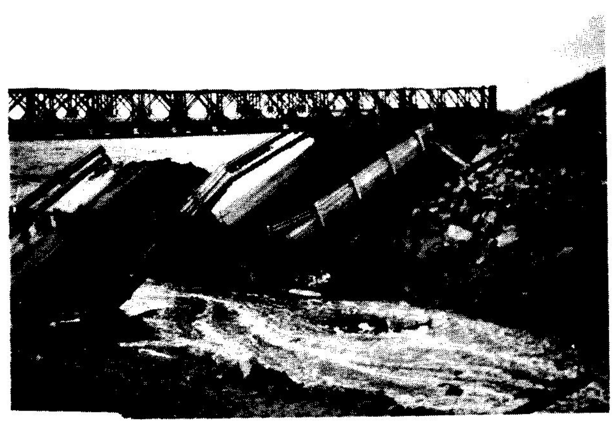


Fig. 13

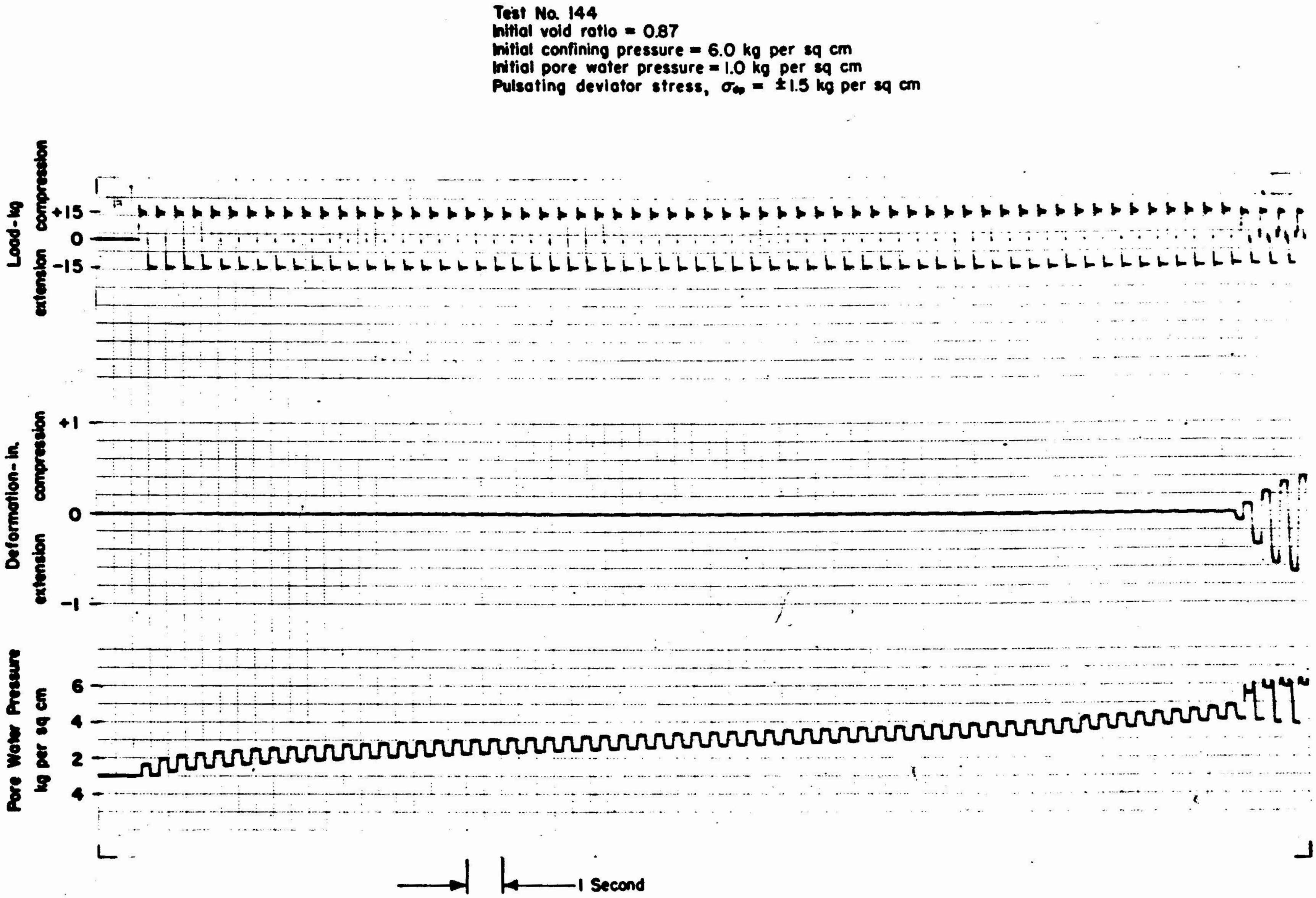



Fig 14 – RECORD OF A TYPICAL TEST ON LOOSE SAND AT LOW PULSATING DEVIATOR STRESS.

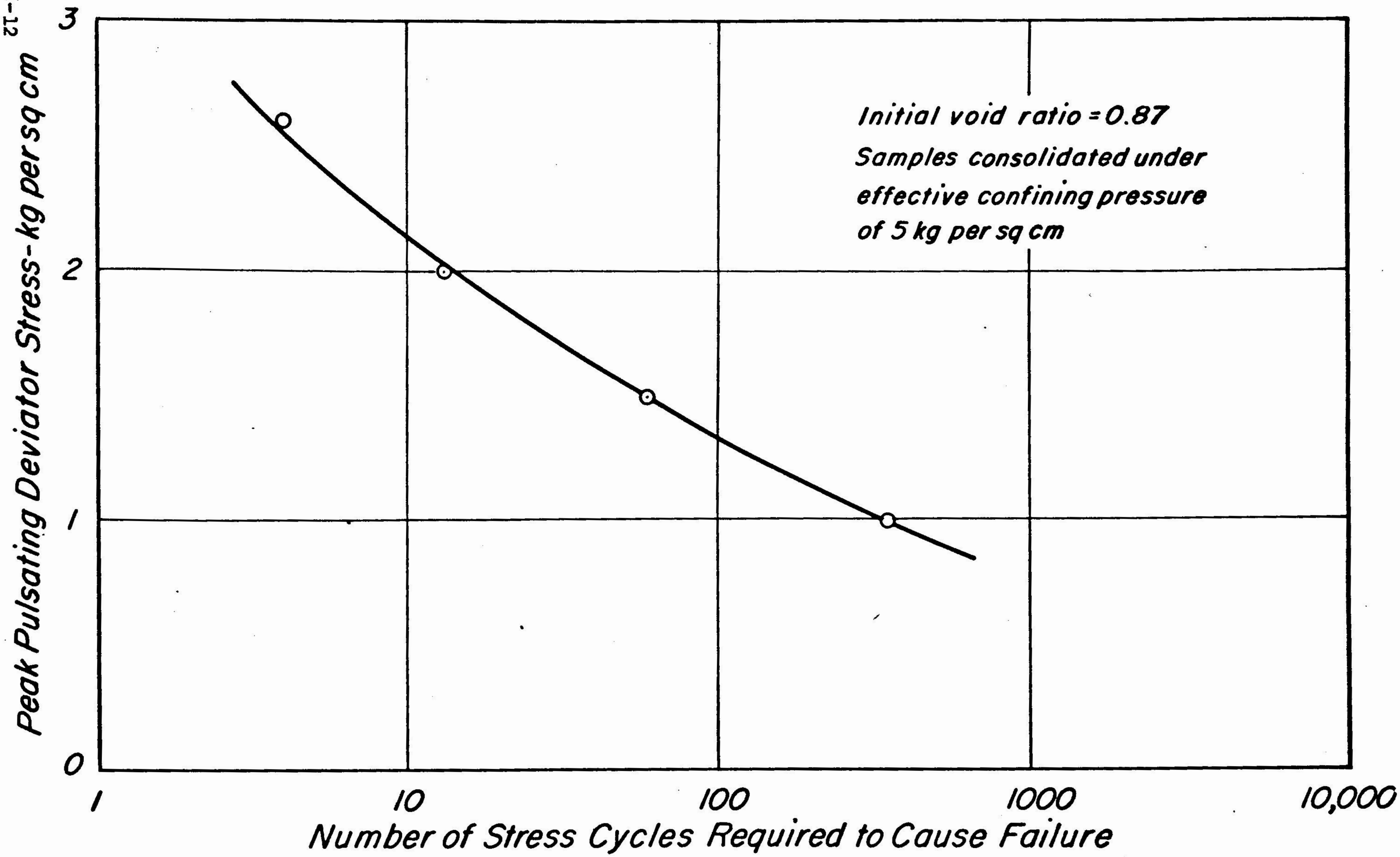



Fig. 15- RELATIONSHIP BETWEEN CYCLIC STRESS AND NUMBER OF STRESS CYCLES CAUSING FAILURE.

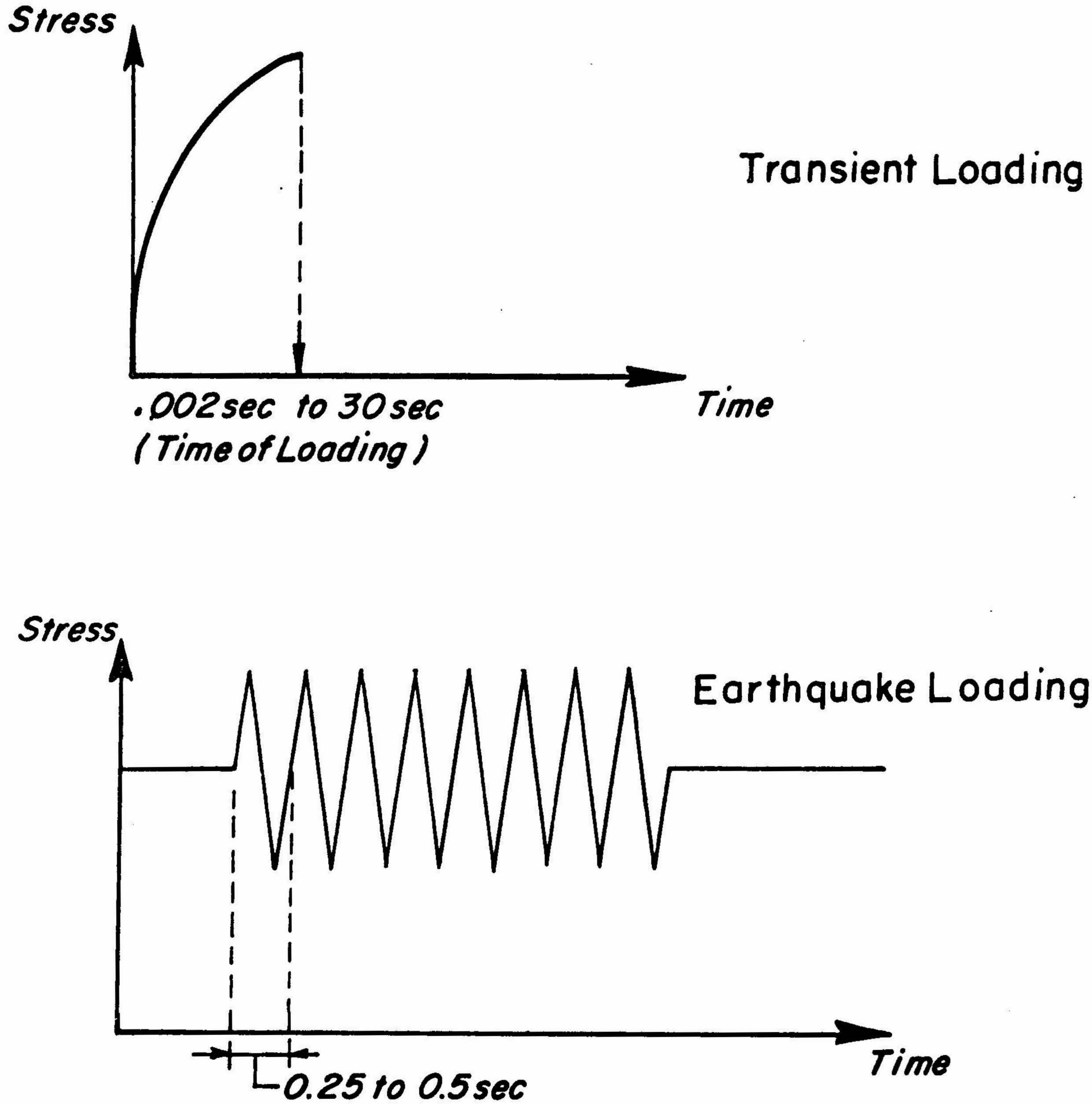



Fig.16- TYPES OF DYNAMIC LOADING

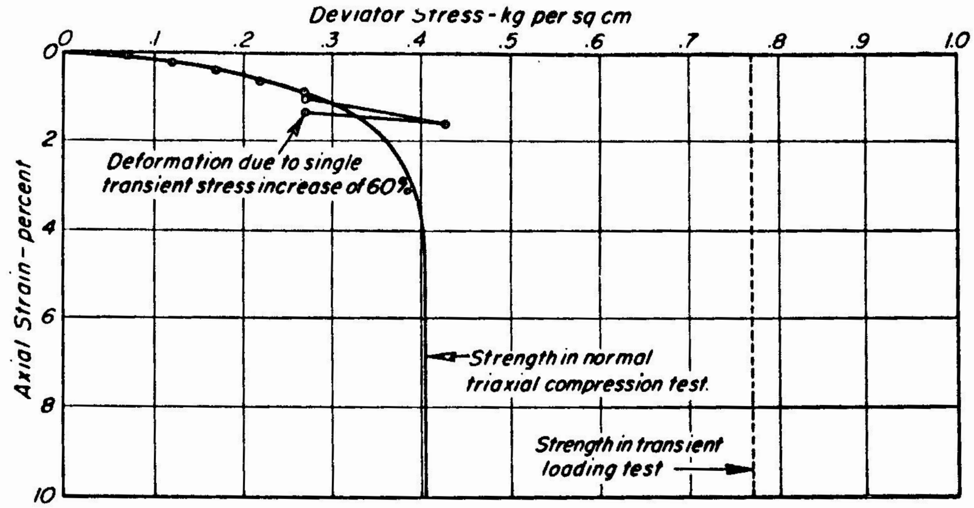



**Fig.17a - SOIL DEFORMATION DUE TO A SINGLE TRANSIENT STRESS PULSE**

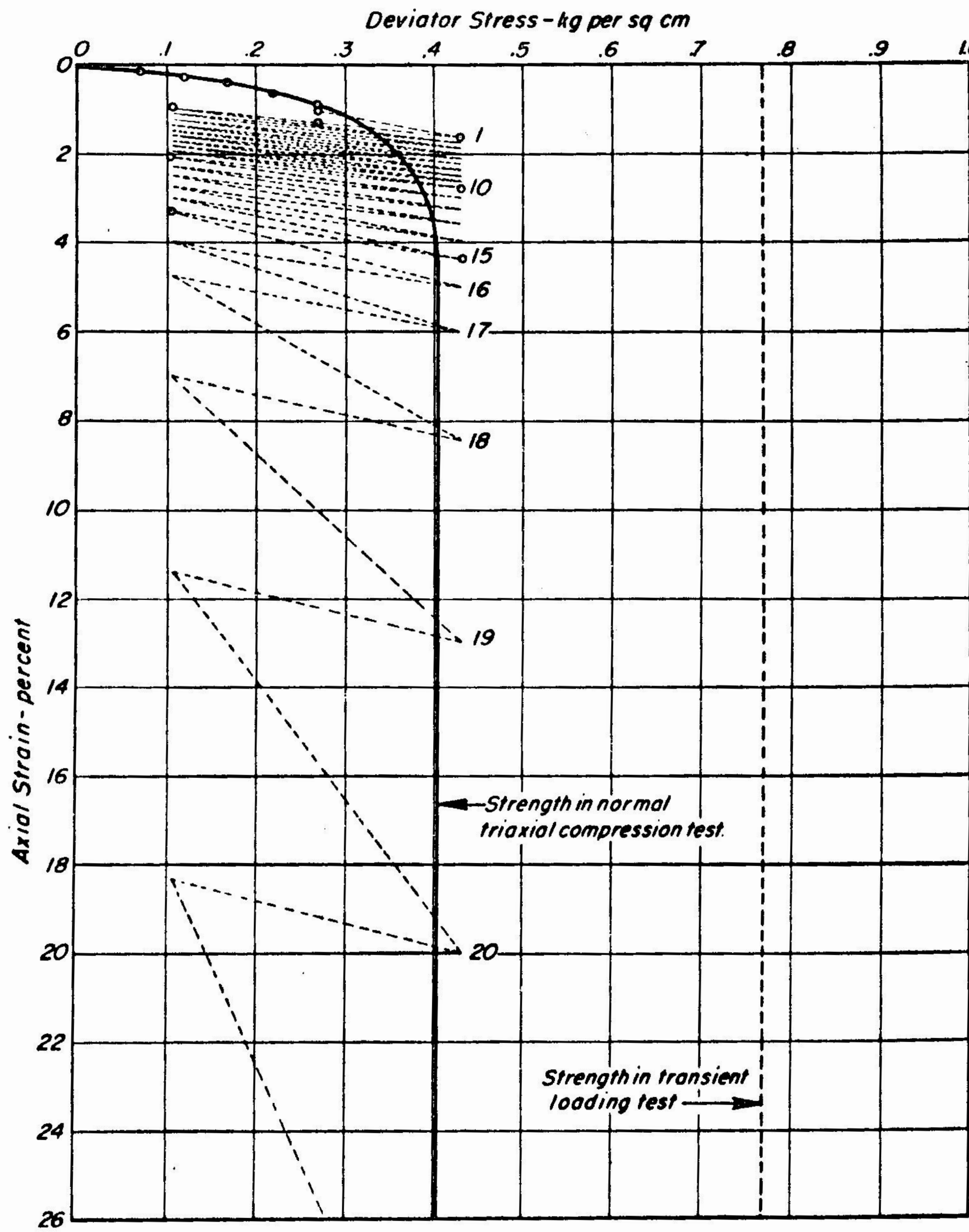





**Fig.17b-SOIL DEFORMATION DUE TO A SERIES OF TRANSIENT STRESS PULSES.**

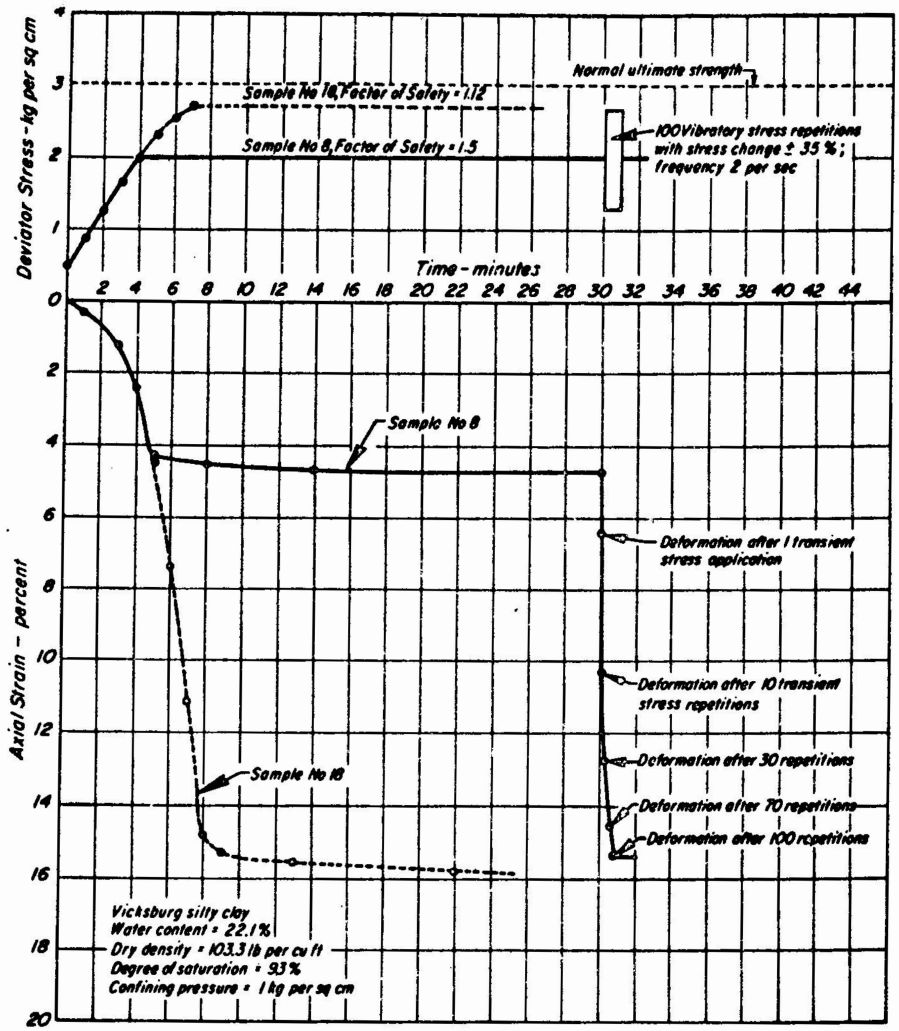



**Fig.18 - CHANGES IN STRESS AND DEFORMATION WITH TIME DURING SIMULATED EARTHQUAKE LOADING TEST ON SILTY CLAY.**

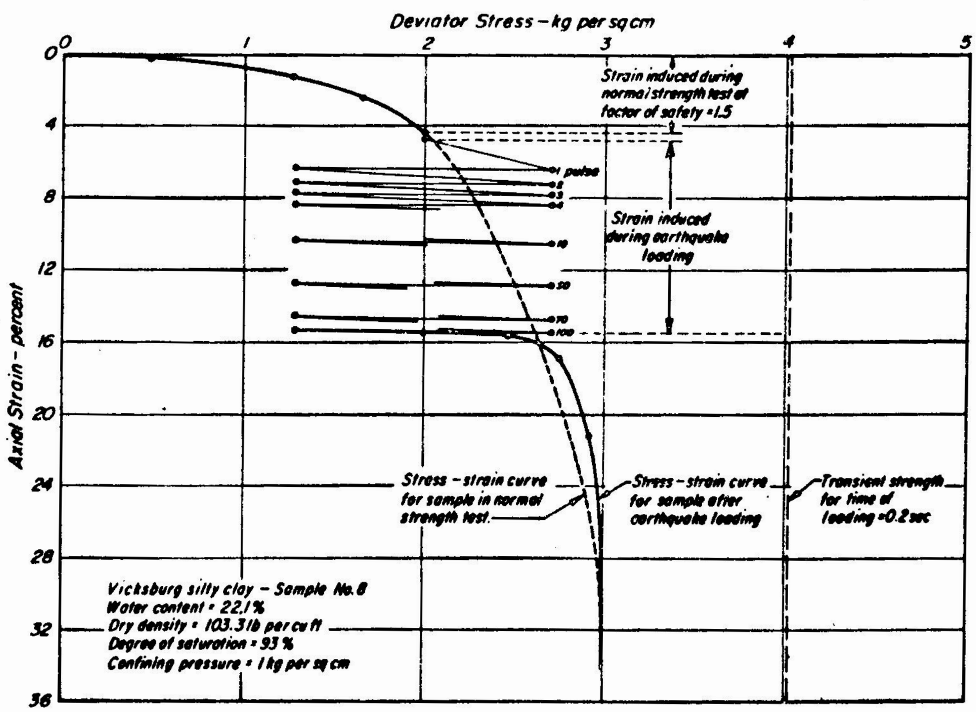
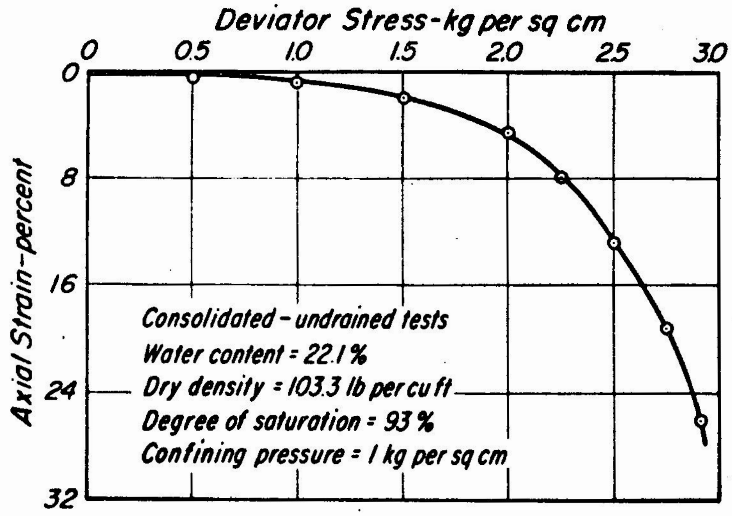



**Fig.19 - STRESS vs STRAIN RELATIONSHIP IN SIMULATED EARTHQUAKE LOADING TEST ON SILTY CLAY.**




Typical stress vs strain relationship
for compacted silty clay.

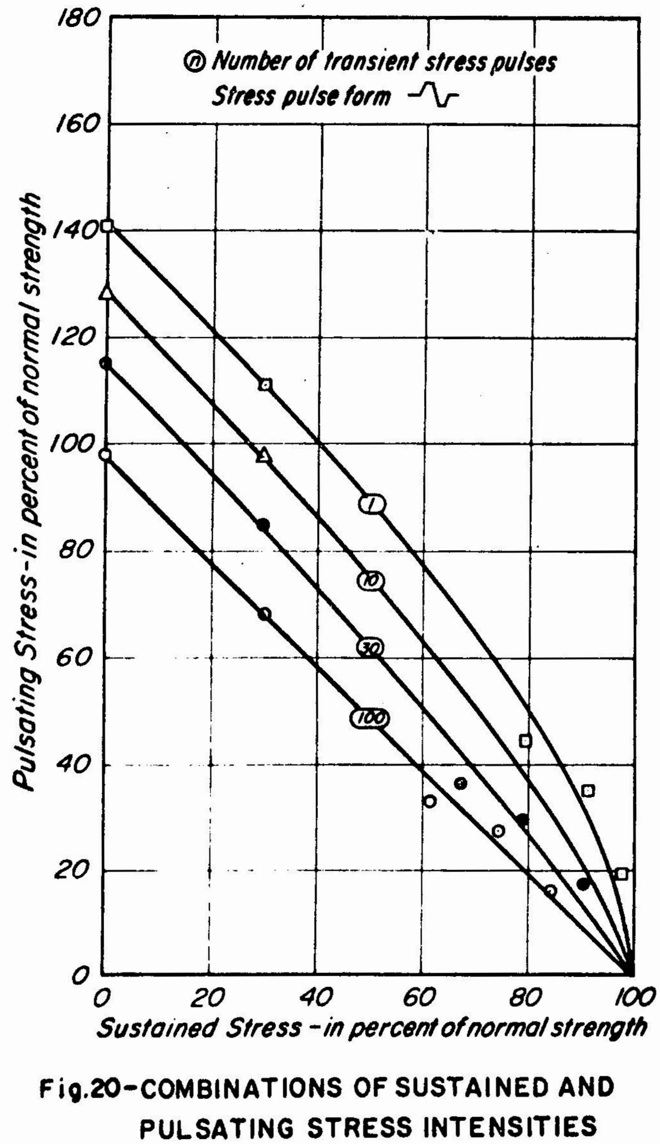



Fig.20-COMBINATIONS OF SUSTAINED AND PULSATING STRESS INTENSITIES CAUSING FAILURE - COMPACTED VICKSBURG SILTY CLAY.