THE EFFECTS OF EARTHQUAKE LOADINGS ON SOIL BEHAVIOUR

by

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INTRODUCTION

Recently a significant amount of attention has been focused on earthquake forces and the general seismicity of the Caribbean region. This has led to the formulation of a CCEO Draft Seismic Code. Based on the records of seismic activity, it has been recommended that the Caribbean region be treated as Zone 3 of the United States Uniform Building Code.

It is generally accepted that local geology and soil conditions play an important part in the seismic effects on structures. This paper presents the general status of knowledge of the effects of seismic forces on the engineering properties of soil. It is the product of a relatively intensive literature search. It is hoped that this paper will permit an understanding of soil behaviour under seismic loadings so that the effects of localised soil conditions may be catered for with more confidence.

EARTHQUAKE GROUND MOTION

Earthquake loadings differ from other dynamic or cyclic loadings as shown on Figs. 1, 2 and 3.

Earthquake characteristics are measured by a seismograph which produces an accelerogram as shown on Fig. 2 from which the ground velocity and displacement can be calculated.

Fig. 2 shows only one component of ground acceleration. There would also be an E-W component and vertical component.

Earthquakes are thought to be generated by stress relief along

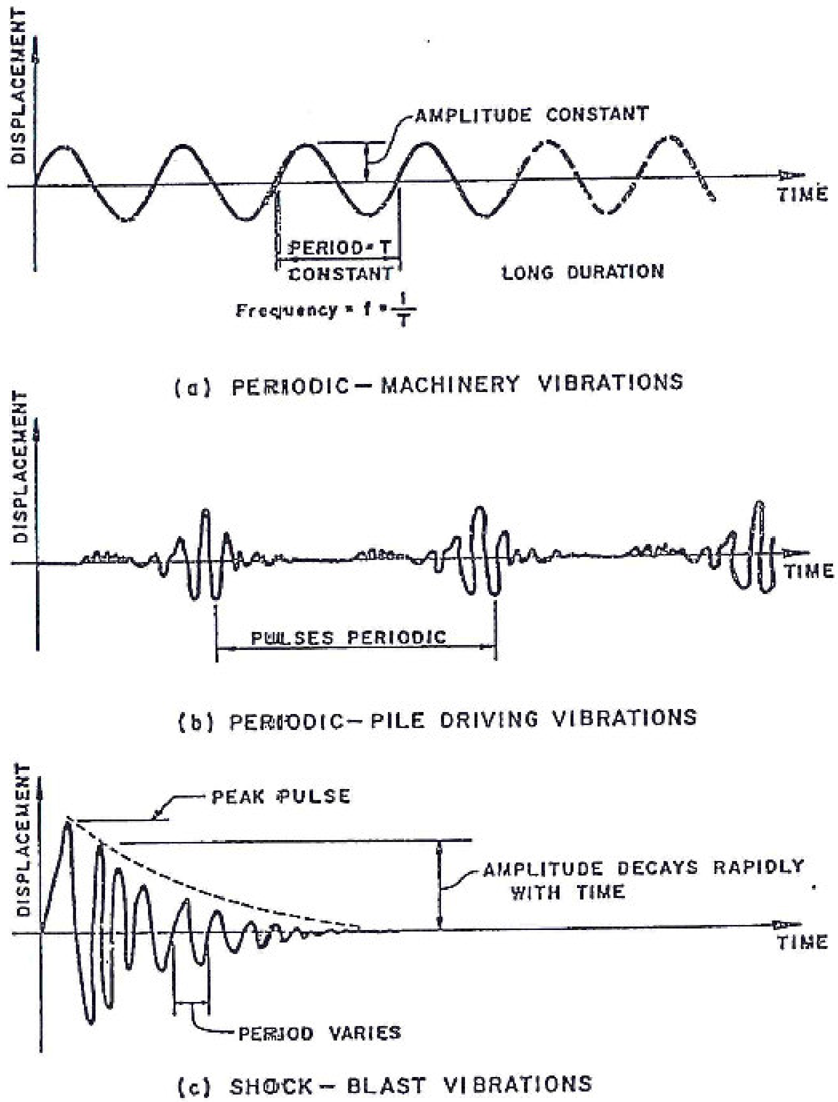

FIG. 1—TYPICAL PERIODIC AND SHOCK GROUND MOTIONS (1)

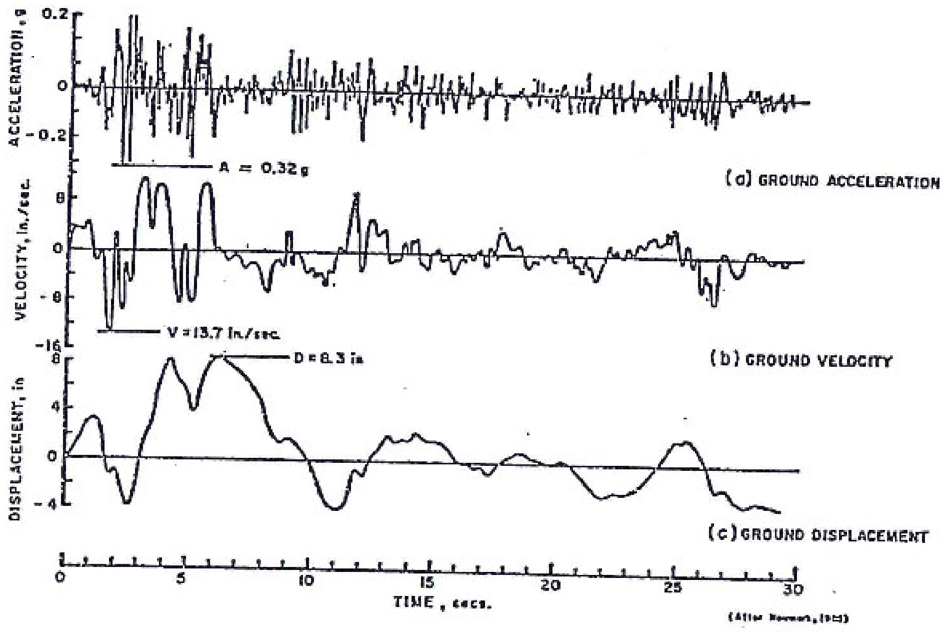

FIG. 2 -STRONG GROUND MOTION, EL CENTRO CALIFORNIA EARTHQUAKE OF MAY 18, 1940, N-S COMPONENT (1)

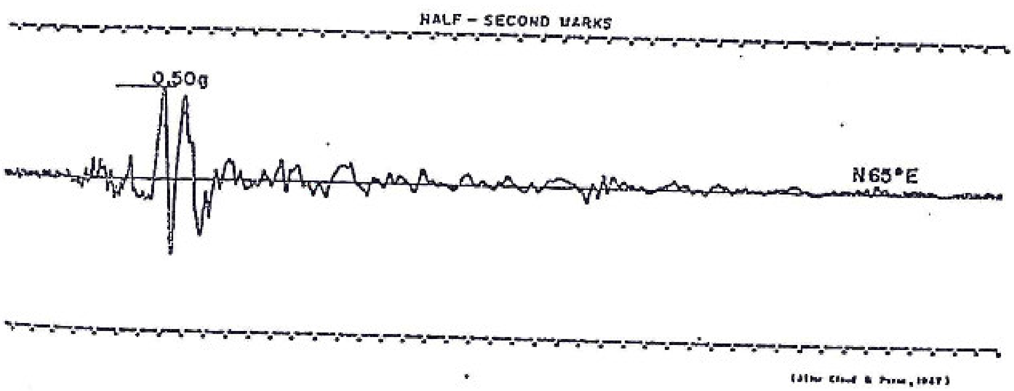

FIG. 3.–ACCELEROGRAM OF HORIZONTAL GROUND MOTION, PARKFIELD EARTHQUAKE JUNE 27, 1966 (1)

faults (2). These cause shear waves to pass upward through the soil which is usually less stiff and has greater damping capacity than the underlying rock. The amplification or attenuation depends not only on the stiffness and damping of soils but also on the earthquake accelerogram and thickness of the various soil layers. The waves significantly attenuate with increasing distance from the source of the earthquake resulting in ground surface motions of more uniform intensity and relatively low frequency.

The maximum horizontal ground acceleration reported in a large magnitude earthquake is 33% g (M = 7.1, El Centro 1940) (2). Several smaller earthquakes have had higher accelerations. Consideration of the maximum energy release on faults indicates that the maximum expected ground accelerations approach 50% g. Maximum ground accelerations of 40% were measured in the San Fernando, California Earthquake (magnitude 6.6) of 9th February 1971 (3). Ground accelerations in the range of 50% - 75% g with several high frequency peaks of 100% g were measured at Pacoema Dam during this earthquake. However, these latter results are not reliable since fracturing and faulting occurred at the recording station.

It is known that surface ground motions are influenced by local geology. The factors that can influence the surface ground motions are (2):

(a) the nature of the source mechanism: the dimensions and orientation of the slipped area of the fault, the stress drop, the nature of the fault movement, its amplitude, direction, time and history.

(b) the travel path of the seismic waves: the physical properties of the rock, discontinuities, layering etc.

(c) local geology: physical properties of the rock and soil layers, vertical and horizontal extent of soil and rock strata, orientations of bedding planes.

In some cases the nature of the soil deposit can be the dominant influence on surface ground movements. Evidence has been presented

(4) that local geology and soil conditions have a more noticeable influence on very weak ground motions than on motions of potentially destructive influence. Thus is demonstrated in Tables 1 and 2 below and in Fig. 4.

Max. acceleration of underlying rock (g's)	Computed amplification factor
0.005	4
0.02	3
0.05	1.4
0.08	0.8
0.13	0.6

TABLE 1: Effect of Magnitude of Rock Motions on Amplification Factor at Alexander Building Site, San Francisco[4]

Type of earthquake	Max. acc. in underlying rock (g's)	Max. ground surface acc. (g's)	Amplification factor
Strong local shock	0.13	0.08	0.6
Great distant earthquake	0.13	0.13	1.0

TABLE 2: Ground Accelerations[4]

Table 1 shows that as the acceleration in the base rock increases the amplification factor decreases becoming less than 1. Table 2 shows that the characteristics of the base rock motions are of great significance in determining ground surface response. The data presented above is based on analyses using lumped mass representation of layered systems (4). The general relationships between base

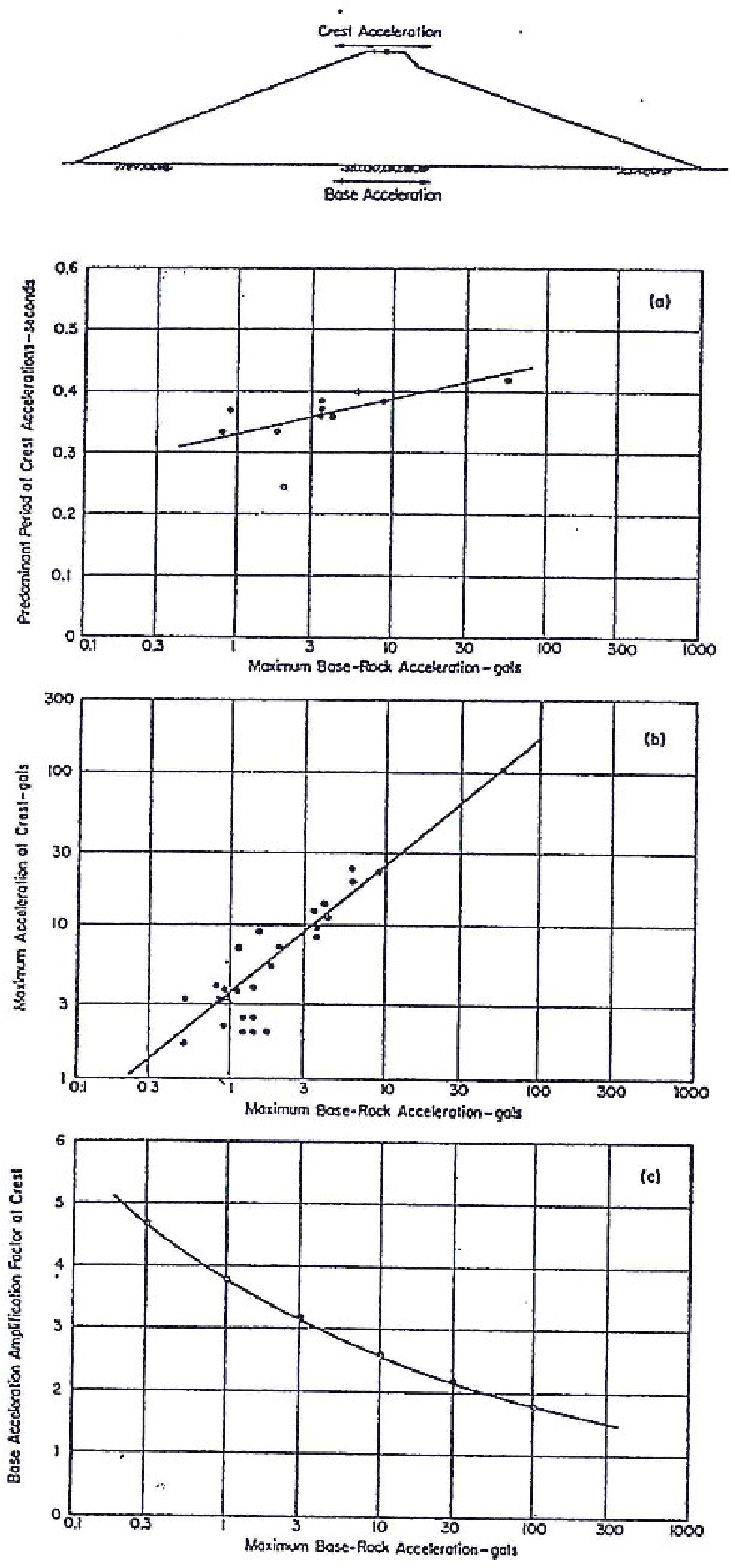

FIG. 4 —OBSERVED RESPONSE OF SANNOKAI DAM DURING EARTHQUAKES (4)

rock motion and amplification factor applicable to soil response above were supported by measurements on the base and crest accelerations of the Sannokcai Dam in Japan (5) as shown in Fig. 4.

The above tends to support earlier thinking, i.e. that the maximum probable significant ground accelerations are likely to approach 33% g and the maximum possible ground accelerations are likely to approach 50% to 60% g (6).

However, the most important aspect of ground motion on soil behaviour is the number of significant acceleration cycles or pulses and not the maximum or peak accelerations as will be shown below.

EFFECT OF GROUND SHAKING ON SOIL

The effect of ground shaking on soil properties depends upon

(a) the magnitude of the significant acceleration pulse.

(b) the number of significant pulses per second

(c) the duration of significant pulses.

For example ground shaking of the type shown in Fig. 2 can be expected to have a much more significant effect on soil properties and structural damage than that shown in Fig. 3.

There is evidence that the effects of vertical shaking are negligible unless the accelerations approach 100% g when the dynamic stresses are small and that the effect of vertical vibrations can normally be neglected (7). On the other hand, the shear stresses associated with horizontal accelerations tend to produce the significant effect on and changes in soil properties.

Seismograph records of ground motions during earthquakes indicate that during the main shock the ground is subject to a horizontal acceleration that may approach its peak intensity as many as 10-15 times in approximately 30 secs. Including after shocks, the total number of larger acceleration pulses can be as great as 30 or 40.

EFFECT OF GROUND SHAKING ON SOIL STRENGTH

Earthquake loadings cause a series of pulsating transient loadings which reverse their direction several times during the event. Differentiation between effects of single transient loadings and cyclic loadings on soil strength should be made. The following ratios between transient strength and static strength have been reported (8).

For sands: $\frac{\text{transient strength}}{\text{static strength}} = 1.1 - 2.0$

For cohesive soils: $\frac{\text{transient strength}}{\text{static strength}} = 1.5 - 2.0$

However, coarse, dry or saturated cohesionless soils do not appear to display an increase in maximum shear resistance with increased rate of strain.

The variation in the ratios above is due to the strain rate. At higher strain rates the higher ratios are observed. It can be seen that soils will normally sustain higher short term loadings than indicated by their normal static strength. Earthquake loadings, however, impose cyclic loads and this effect is different to that of single transient loadings.

EFFECT OF CYCLIC LOADING ON CLAY STRENGTH

Measurement of the effects of cyclic or pulsating loads on the shear strength of saturated clays (9) were made in the triaxial cell using a pulsating deviator stress. Both static and transient (0.2 secs) strengths were measured. Cyclic loadings were applied in both one direction and two directions of loading.

Samples of silty clay were first subjected to a sustained stress less than the normal static compression strength until equilibrium was established. The samples were then subjected to several cycles of pulsating deviator stress. As can be seen in Fig. 5, the pulsating deviator stress induced permanent strains in the sample which increased with the number of cycles. After 100 cycles of pulsating stress the sample was loaded to failure which corresponded closely to the normal static strength of the soil.

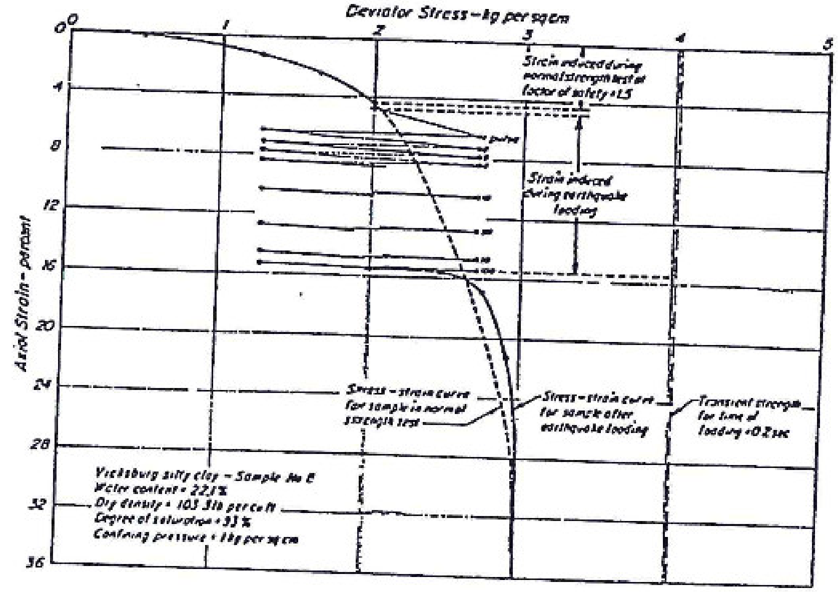

FIG. 5 –STRESS VS STRAIN RELATIONSHIP IN SIMULATED EARTHQUAKE LOADING TEST ON SILTY CLAY (9)

The magnitude of the deformations occurring in such a test depends upon

(a) the magnitude of sustained stress,

(b) the magnitude of superimposed pulsating stress, and

(c) the number of stress pulses applied.

Fig. 6 shows the above effects.

Also investigated were the effects of one directional and two directional loading, confining pressure and the nature of the stress pulse.

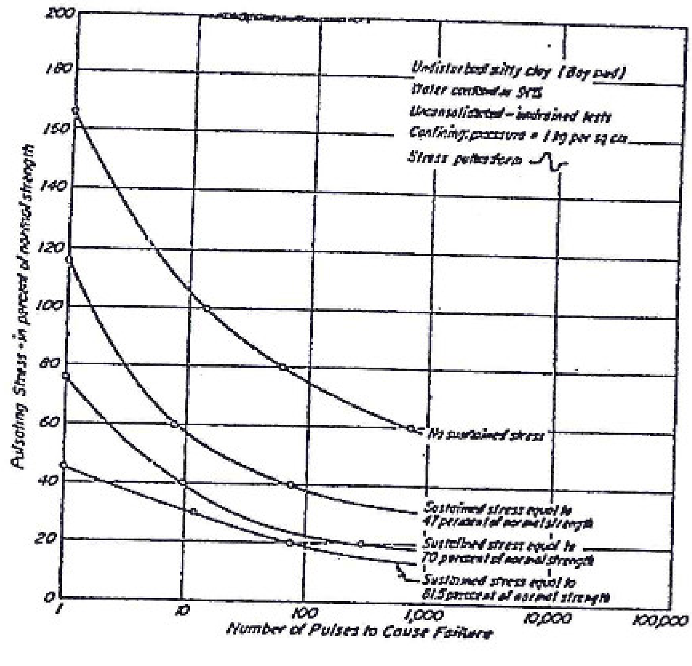

FIG. 6 —RELATIONSHIP BETWEEN STRESS LEVEL AND NUMBER OF PULSES CAUSING FAILURE (9)

It was found that the effect of the stress pulse form and the difference between one directional and two directional loading was only significant at low levels of sustained stress. These effects could then be ignored where clay soils are already loaded with conventional factors of safety (i.e. where 50%-67% of the shear strength is mobilised). It was also found that the effects of principal stress ratio during consolidation and confining pressure were small and did not affect the accuracy of the test results.

The amount of strain after each cycle of pulsating stress at the various sustained loadings was measured and was used to develop the curves shown in Figs. 7 and 8 for two different types of clays. Both figures show surprisingly similar results. The figures show the strain envelope vs. sustained plus pulsating stress at 10 and 100 cycles for

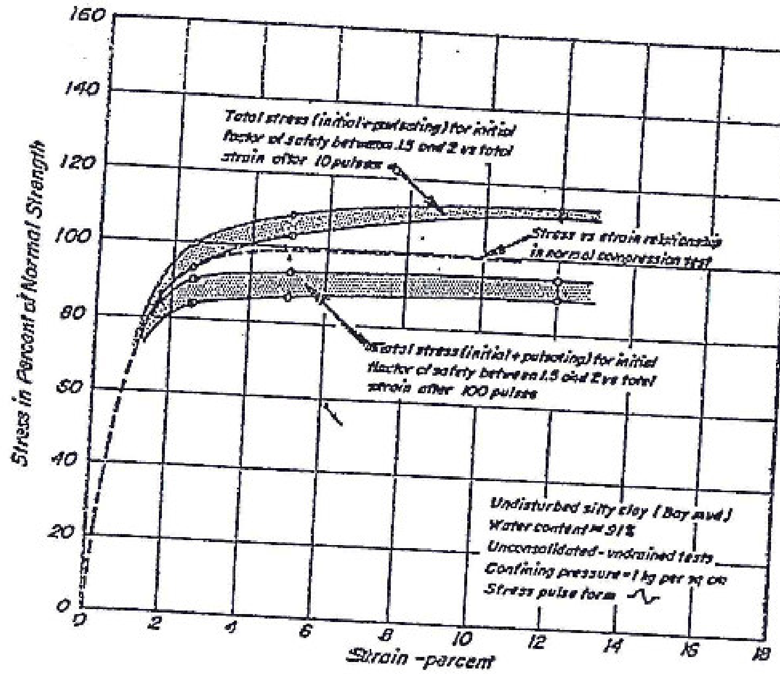

FIG. 7 —RELATIONSHIPS BETWEEN TOTAL STRESS AND TOTAL STRAIN UNDER PULSATING LOAD CONDITIONS—SAN FRANCISCO BAY MUD (9)

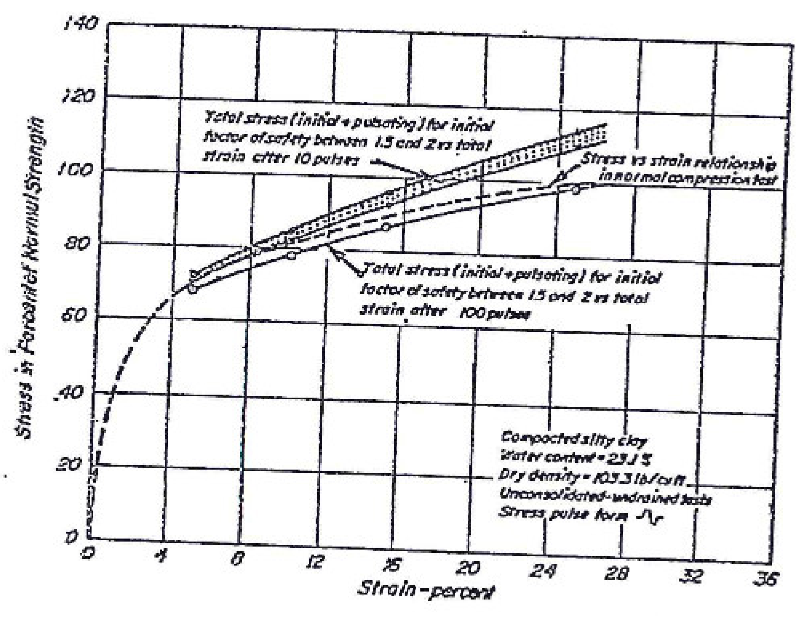

FIG. 8. —RELATIONSHIPS BETWEEN TOTAL STRESS AND TOTAL STRAIN UNDER PULSATING LOAD CONDITIONS—VICKSBURG SILTY CLAY (9)

samples with sustained stress of 50% and 67% of the normal static strength (i.e. factor of safety = 2.0 and 1.5). Also plotted is the normal static compression stress strain curve which agrees remarkably well with the above plots and corresponds to approximately 30 pulses; the likely number of pulses in a large magnitude earthquake.

The conclusion is therefore that the normal static compression curve may be used to evaluate the factor of safety of saturated cohesive soils under earthquake loadings where conventional factors of safety and total strength analyses are used.

It should be noted here that the above could not be expected to apply to strain softening materials such as over-consolidated clays.

PORE WATER PRESSURES DEVELOPED DURING CYCLIC LOADING OF CLAYS

The above investigation (9) also included testing to determine the effect of pulsating stresses on pore water pressures in saturated clays.

In Fig. 9 an undisturbed sample of sensitive clay was subjected to a sustained loading of 65%. After one day it was subjected to 100 dynamic stress pulses of ± 50%. The pulses caused an immediate increase in axial strain of approx. 5.5% but did not cause failure. On restoration of the original stress conditions the sample deformed an additional 12% over the following 12 days and failed completely after 13½ days.

These results were said to suggest that in sensitive clays an earthquake does not in itself cause failure but may initiate an increase in pore pressure and creep deformations which could lead to failure after some period of time. This delayed movement of slopes after earthquakes has been observed in areas of soft sensitive clays.

On the other hand, it has been shown by other investigators (10) that neither the creep strength nor strength after creep of samples of the same sensitive clay were reduced by seismic loadings. It was concluded that the most hazardous period for homogeneous deposits which are creeping are during the earthquake because the deformations may accumulate rapidly and possibly immediately after the earthquake

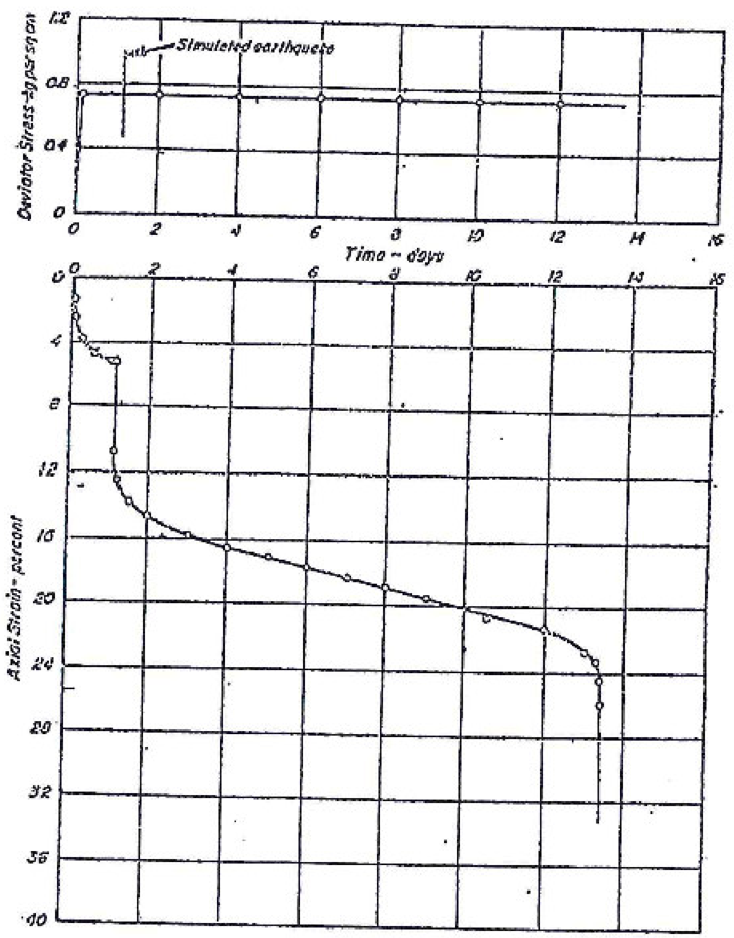

FIG. 9 —LOSS IN STRENGTH BECAUSE OF SIMULATED EARTHQUAKE LOADING—SAN FRANCISCO BAY MUD (9)

because rates of deformation may then be large. It is suggested that liquefaction of intercalcated sand lenses could cause what might appear from their surface expression as large creeping movements. It should be noted that in Fig. 9 the number of pulses were significantly in excess of those in (10) and of those to be expected in a seismic event.

EFFECT OF CYCLIC LOADING ON THE SHEAR STRENGTH OF SANDS

The effect of cyclic loads on fine grained sand has been the subject of several investigations (11) (12) (13) (14) (15) and (16) in order to understand the most dramatic aspect of earthquake loading on soil, i.e. liquefaction. Liquefaction is described as occurring when the induced pore water pressures due to cyclic loading become equal to the confining pressure on the soil. Under such conditions a soil can loose its strength and acquire a degree of mobility during an earthquake sufficient to permit movements ranging from several feet to several thousand feet. The results of these investigations show that critical void ratio concepts as proposed by Casagrande (17) do not apply to cyclic loadings.

The effects of cyclic loadings on dry sands, though not as dramatic and of great concern as those on saturated sands, are worth mentioning. The results of tests on uniform sands in (12) show that if the principal stress ratio under vibration is kept below a certain level the shear strain is only slightly greater than that obtained in a static test. The magnitude of the principal stress ratio above which large strains may develop under vibration is dependant upon the confining pressure. At low confining pressures (7 psi) this ratio is about ½ the principal stress ratio in static tests. At higher confining pressures (20 psi) it is about ¾ of the static ratio.

The most comprehensive results of tests on saturated sands may be found in (11). Samples of fine to medium sand at various densities and confining pressures were subjected to undrained pulsating loads in a triaxial cell.

The results of the tests on loose sand are shown in Fig. 10. It will be seen that the sample of sand showed no noticeable deformation until the ninth cycle when sudden large strains exceeding 20% were experienced. This occurred once the pore pressure became equal to the confining pressure which may be defined as liquefaction.

The important conclusion here is that liquefaction occurs very suddenly. The following conclusions with regard to liquefaction can be-made (11).

(a) For a given density and confining pressure the number of stress cycles required to cause liquefaction decreases as the magnitude of the cyclic deviator stress is increased.

(b) The higher the confining pressure the greater the number of stress cycles required to produce liquefaction.

(c) As the density of sand increases the amount of stress cycles to cause liquefaction increases. In addition the strain amplitude after liquefaction is less for dense samples than for loose samples: Dense samples of sand did not show the sudden large increases in strain on liquefaction but displayed a gradual increase in strain with increased cycles of deviator stress.

(d) The application of static stress after liquefaction will produce significant displacement but a simultaneous development of a high resistance to further deformation will occur. The amount of resistance to displacement increases and the amount of displacement decreases as the density increases.

LIQUEFACTION

Based on the above the danger of liquefaction of a saturated sand depends upon:

1. the void ratio of the sand - the higher the void ratio the more easily liquefaction will occur.

2. the confining pressure acting on the sand - the lower the confining pressure the more easily liquefaction can occur.

3. the magnitude of the cyclic stress or strain - the larger the cyclic stress or strain the lower the number of cycles to induce liquefaction.

4. the number of stress cycles to which the sand is subjected.

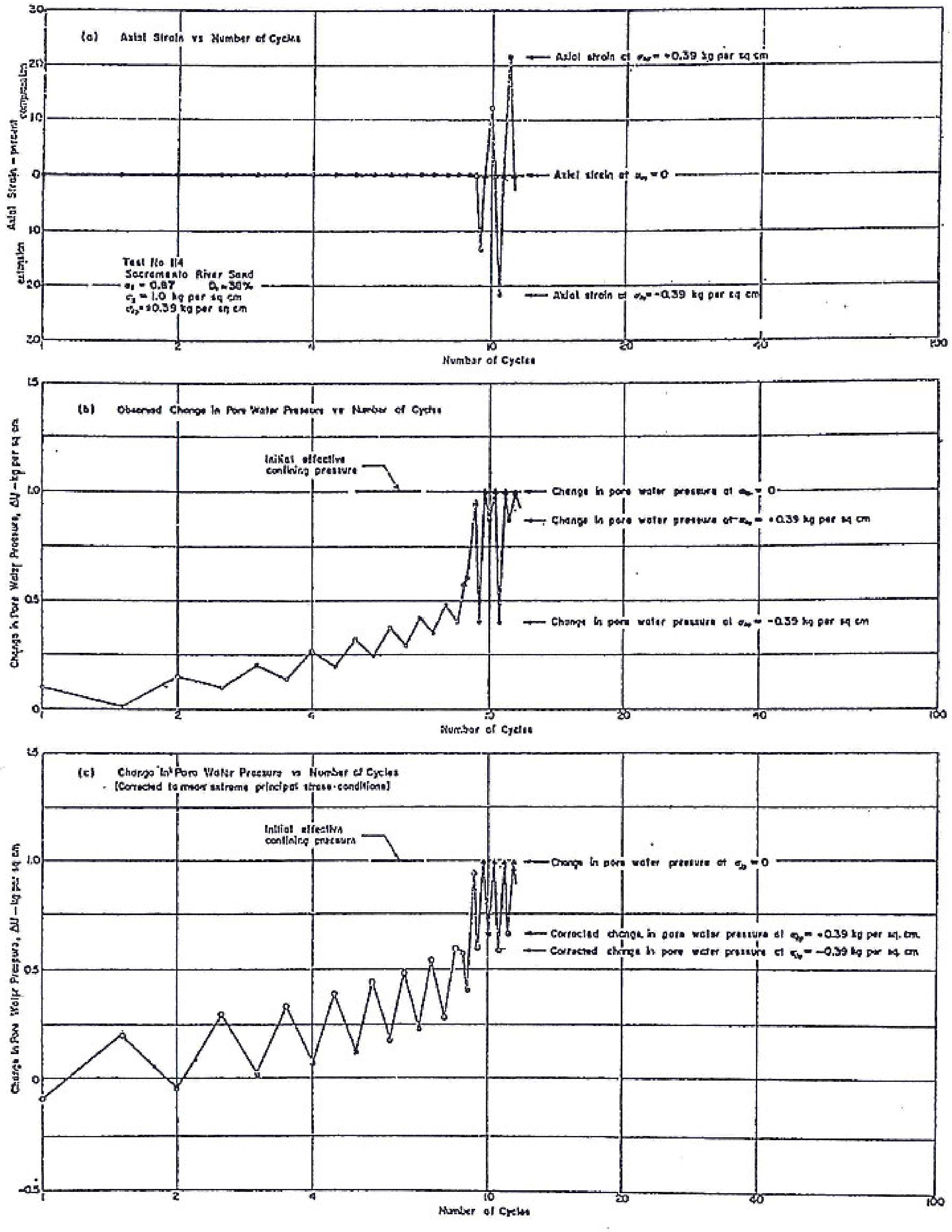

FIG.10—TYPICAL PULSATING LOAD TEST ON LOOSE SAND (11)

In addition grain size characteristics are important. Well graded, medium to fine sand having 10% or more particles in the silt size are less likely to liquefy than sand having the same average grain size particles but poorly graded and-having no fines.

Based on the experience of Niigata earthquake of 1964 the following general conditions for liquefaction have been qualitatively documented (1).

1. The percentage of silt and clay sizes should be less than 10%.
2. The particle diameter at 60% passing should be between 0.2 mm and 1.0 mm.
3. The uniformity coefficient $\frac{(D_{60})}{(D_{10})}$ should be between 2 and 5
4. The blow count (N) in the Standard Penetration Test should be less than 15. In the heavily damaged zone the blow counts were N = 7.

In general, experience suggests that liquefaction might occur for soils having a relative density less than 50% during ground motions with accelerations in excess of 0.1 g. For relative densities-greater than 75% liquefaction for most earthquake loadings is unlikely.

The experience of Niigata and theoretical calculations indicate that the depth to the ground water table is important. For example, it is reported (18) that if the depth to the ground water table in the heavily damaged zone of Niigata was 5 ft. lower only minor damage may have been experienced.

It should be noted that the confining pressure is least at the ground surface and increases with depth. Hence liquefaction is likely to occur at shallow depths where the overburden pressures are least i.e. close to the ground surface. Liquefaction at the surface could then spread downwards since the confining pressure on lower layers would be reduced.

Similarly, if the soil density decreased with depth liquefaction may be initiated in a zone below the ground surface. The upward flow of water may then induce liquefaction in the upper layers.

The above indicates an important feature of liquefaction. Its onset in one zone of a deposit can lead to liquefaction of other zones which would otherwise have remained stable.

In addition, a liquefied sand layer will revert to a firm condition on the cessation of cyclic stresses or earthquake ground motion. This is supported by observations of several major slides in Anchorage as a result of the 1964 Alaskan earthquake. Where sliding was along liquified layers of sand, observers report that sliding stopped immediately upon cessation of ground motions.

DETERMINATION OF LIQUEFACTION POTENTIAL OF A SOIL PROFILE

A simplified method of determining the liquefaction danger in a cohesionless soil profile has been presented (16). Comparison with the occurence liquefaction at Niigata during the 1964 earthquake indicates reasonable reliability. However, the method has a number of simplifications and approximations and is not applicable to all soil profiles. Also a major shortcoming is the necessity to develop a definitive failure criterion applicable to field conditions.

The method consists of determining the cyclic shear stresses induced by the design earthquake in the soil profile and comparing these with shear stress required to cause liquefaction as shown on Fig. 11.

DETERMINATION OF THE SHEAR STRESSES CAUSED BY THE DESIGN EARTHQUAKE

Consider a body of soil under a horizontal ground surface having no initial shear stresses and subjected to a maximum horizontal acceleration a_{max} as shown in Fig. 12. If the column of soil of an element of depth 'h' behaved as a rigid body then the maximum shear stress

$$(J_{max})_r = \frac{\partial h \ a_{max}}{g} \qquad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

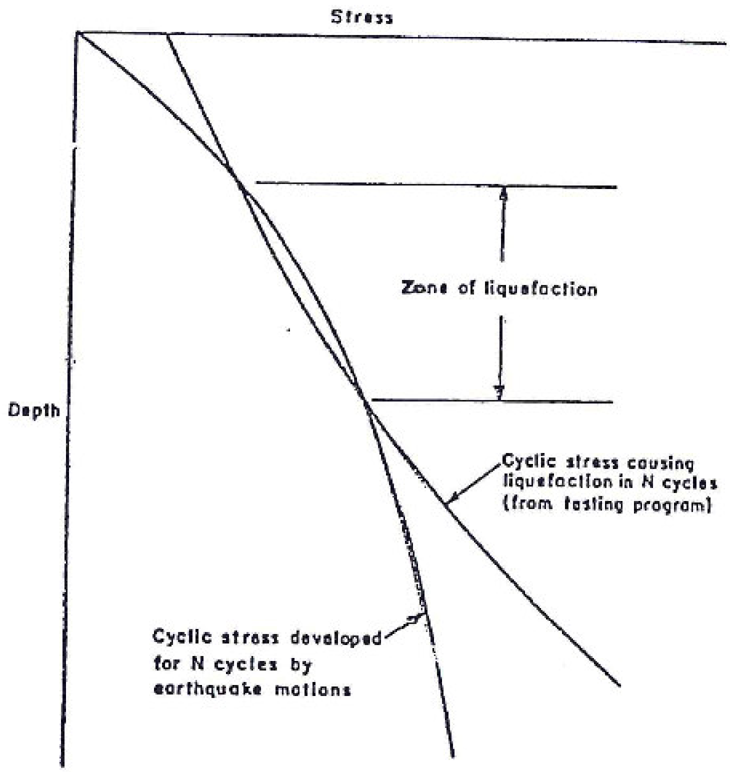

FIG. 11–METHOD OF EVALUATING LIQUEFACTION POTENTIAL (16)

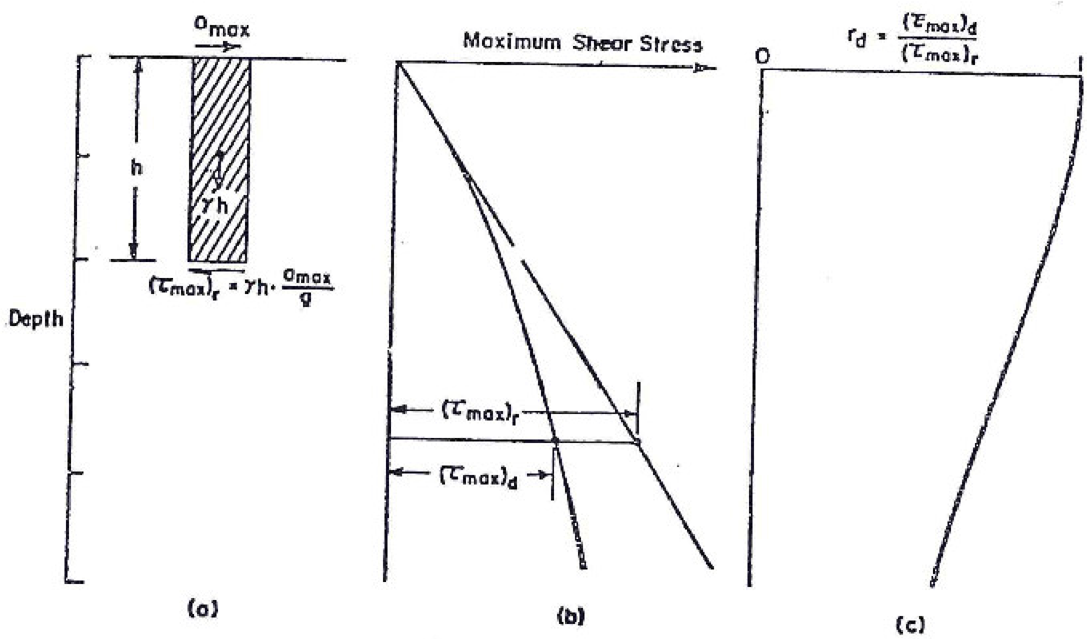

FIG. 12–DETERMINATION OF MAXIMUM SHEAR STRESS (16)

However, since the soil column is not rigid and is deformable, the actual shear stress at depth 'h' $(J_{max})_d$ will be less than $(J_{max})_r$ so that

$$(J_{max})_d = r_d \, (J_{max})_r \quad \ldots\ldots\ldots\ldots (2)$$

where r_d = a stress reduction coefficient less 1

Typical variations in values of r_d are shown in Fig. 13 for a wide variety of earthquake motions and soil conditions having the upper 50 ft. of sand. It may be seen that in the upper 40 ft. the error involved in using the average values shown by the dashed line is not great.

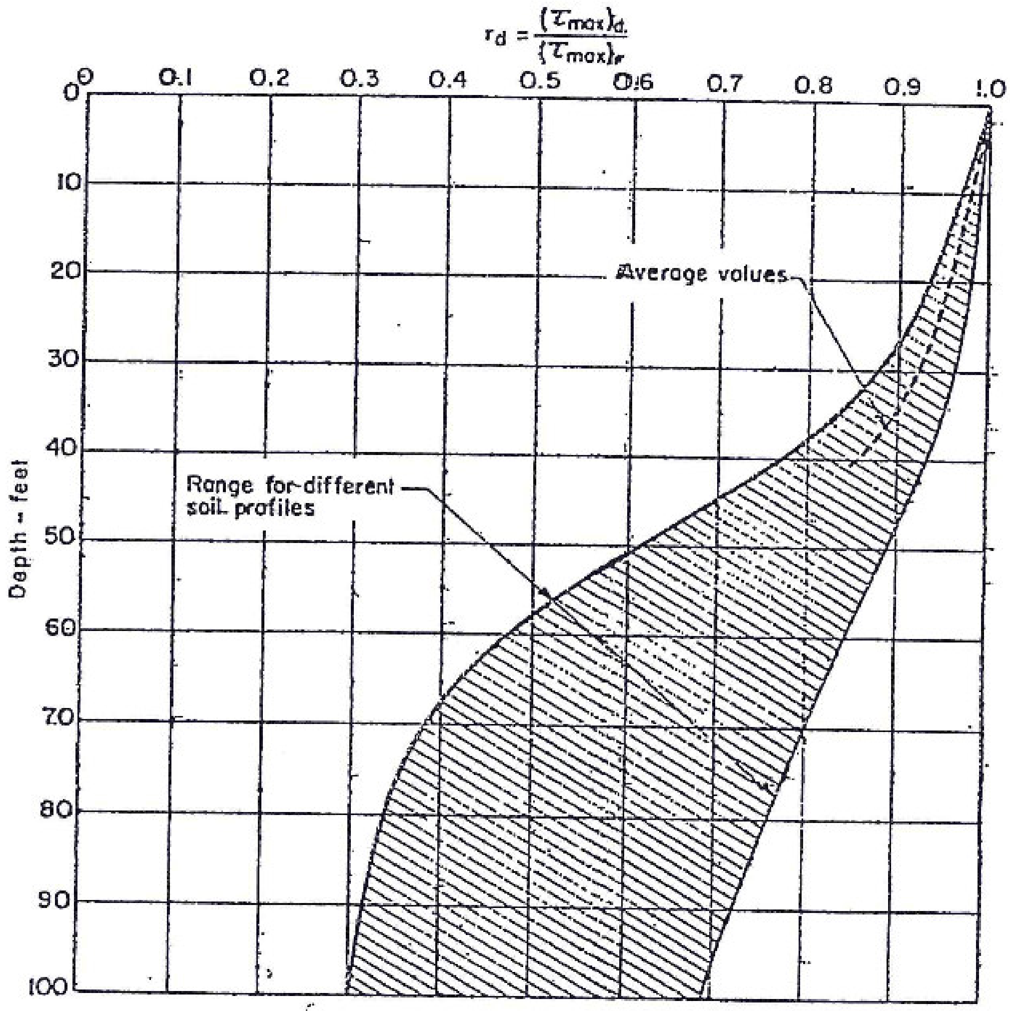

FIG.13-- RANGE OF VALUES OF r_d FOR DIFFERENT SOIL PROFILES (16)

Since the earthquake accelerogram is irregular as shown in Fig. 2, the actual time history of shear stress at any point in a soil deposit will be irregular. It is therefore necessary to determine the equivalent uniform average shear stress. It is suggested that the average equivalent uniform shear stress J_{av} is about 65% of the maximum shear stress.

From the above, the average cyclic shear stress may be deduced from the equation

$$J_{av} = 0.65 \times \frac{\partial h}{g} a_{max} \times r_d \quad \ldots\ldots\ldots\ldots\ldots \quad (3)$$

The appropriate number of significant stress cycles N_c will depend upon the magnitude of the earthquake. Representative numbers of stress cycles suggested are

Earthquake Magnitude	Number of Significant Cycles N_c
7	10
7½	20
8	30

DETERMINATION OF THE SHEAR STRESSES TO CAUSE LIQUEFACTION

The liquefaction of sand by cyclic loading in triaxial tests have been measured by a large number of independent investigators (19). These have been summarised as plots of $\frac{\sigma d_c}{2\sigma a}$ vs. D_{50} at a relative density of 50% for both 10 and 30 cycles (16) in Figs. 14 and 15 respectively. $\sigma\ d_c$ corresponds to the cyclic deviator stress causing failure and $\sigma\ a$ the ambient or cell pressure.

Under field conditions liquefaction is found to depend upon the effective overburden pressure and can be expressed by the ratio

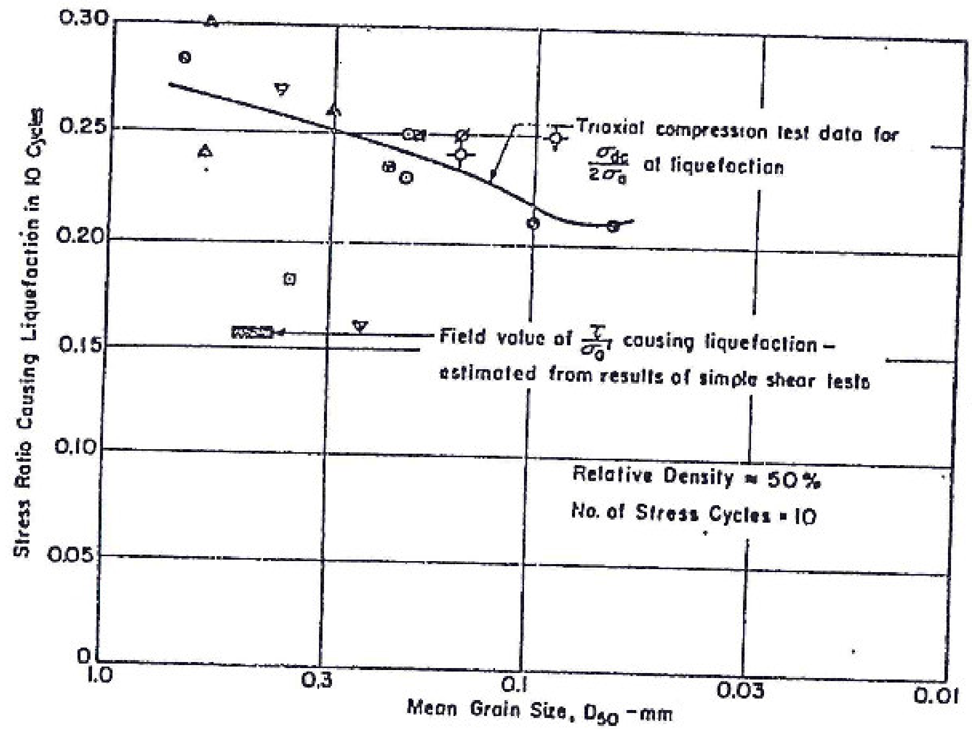

FIG. 14—STRESS CONDITIONS CAUSING LIQUEFACTION OF SANDS IN 10 CYCLES (16)

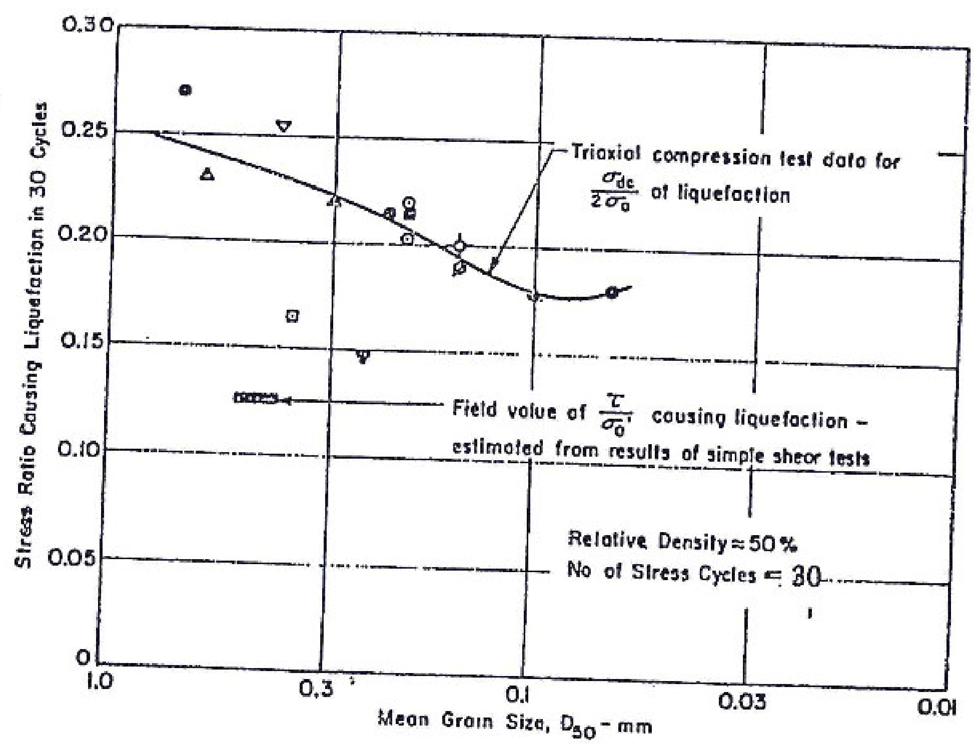

FIG. 15-STRESS CONDITIONS CAUSING LIQUEFACTION OF SANDS IN 30 CYLES (16)

$\frac{J_{hv}}{\sigma'_o}$. J_{hv} corresponds to the cyclic stress causing liquefaction and σ'_o to the effective overburden pressure.

Because the stress conditions in the cyclic tests in the triaxial test are different from the field conditions

$$\frac{J_{hv}}{\sigma'_o} = \frac{\sigma_{dc}}{2\,\sigma_a \text{ triax}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (4)$$

or

$$\frac{J_{hv}}{\sigma'_o} = C_r \frac{\sigma_{dc}}{2\,\sigma_a \text{ triax}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (5)$$

C_r has been found to depend upon the relative density of the sand and may be approximated from Fig. 16.

Based on the above, the field liquefaction potential can be determined from Figs. 14, 15 and 16 for a relative density (D_r) of 50%. The stresses required to cause liquefaction for sands at other relative densities may be estimated from the fact that for relative densities up to approximately 80%, the shear stress required to cause liquefaction is approximately proportional to the relative density (D_r) hence

$$\frac{J_{hv}}{\sigma'_o \text{ field}} = C_r \left.\frac{\sigma_{dc}}{2\,\sigma_a}\right|_{D_r = 50} \times \frac{D_r \text{field}}{50} \quad \ldots\ldots \quad (6)$$

The danger of liquefaction in any deposit can then be estimated from equations (6) and (3) once the grain size characteristics and relative density of the deposit is known and the design earthquake ground acceleration and number of cycles are known.

It should be pointed out that the above method involves a considerable number of approximations and simplifications in dealing with the liquefaction problem. As such it should be used with extreme care. A knowledge of the implications of the approximations and simplifications is required in applying the above to any field situation if meaningful results are desired.

The method has been modified by the authors who developed it to relate liquefaction danger to N values (16). It is felt that the errors involved in relating N values to relative density are much too large to permit liquefaction potential to be determined from N values with any real degree of accuracy. This view is supported by others (20). The relationships between N values and relative density are particularly inaccurate at shallow depths where liquefaction potential is likely to be greatest.

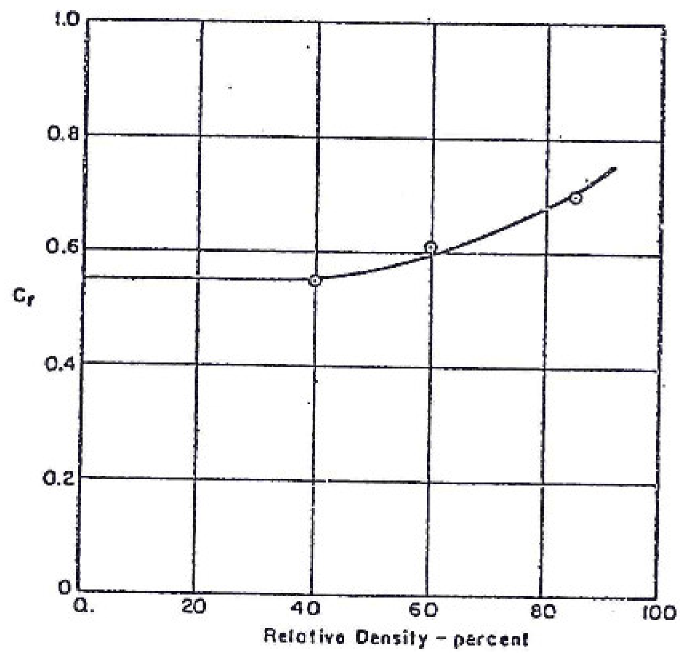

FIG. 16—RELATIONSHIP BETWEEN c_r AND RELATIVE DENSITY (16)

COMPRESSIBILITY OF SOILS UNDER EARTHQUAKE LOADINGS.

Little is known of the effect of cyclic loading on the compressibility of cohesive soils. However the cyclic shear strains induced by earthquakes in soil deposits are capable of causing densification of granular soils (21). The following conclusions have been made regarding dessification of sands under cyclic loadings (7).

1. when the dynamic stresses are small no noticeable densification occurs for accelerations less than about 1 g.

2. when the dynamic stresses small compared to the initial overburden pressure, there is still no noticeable densification.

3. vertical accelerations during earthquakes cause very little densification.

However, the shear stresses associated with horizontal accelerations may produce significant densification and resultant subsidence.

Horizontal cyclic shear strains such as those generated in an earthquake can cause vertical strains as shown in Figs. 17 and 18. The amount of settlement depends upon the magnitude and number of cycles of shear strain, the relative density of the soil and the overburden pressure.

Reference (21) presents a method whereby the amount of vertical compression or settlement of dry or moist sands undergoing earthquake type loadings can be predicted. The type of relationships shown Figs. 17 and 18 were obtained in the Soil Mechanics Laboratory of the University of the West Indies for loose superficial layers of soil at a site close to the University. These relationships were used to perdict settlements under various assumed earthquake loadings for a proposed structure.

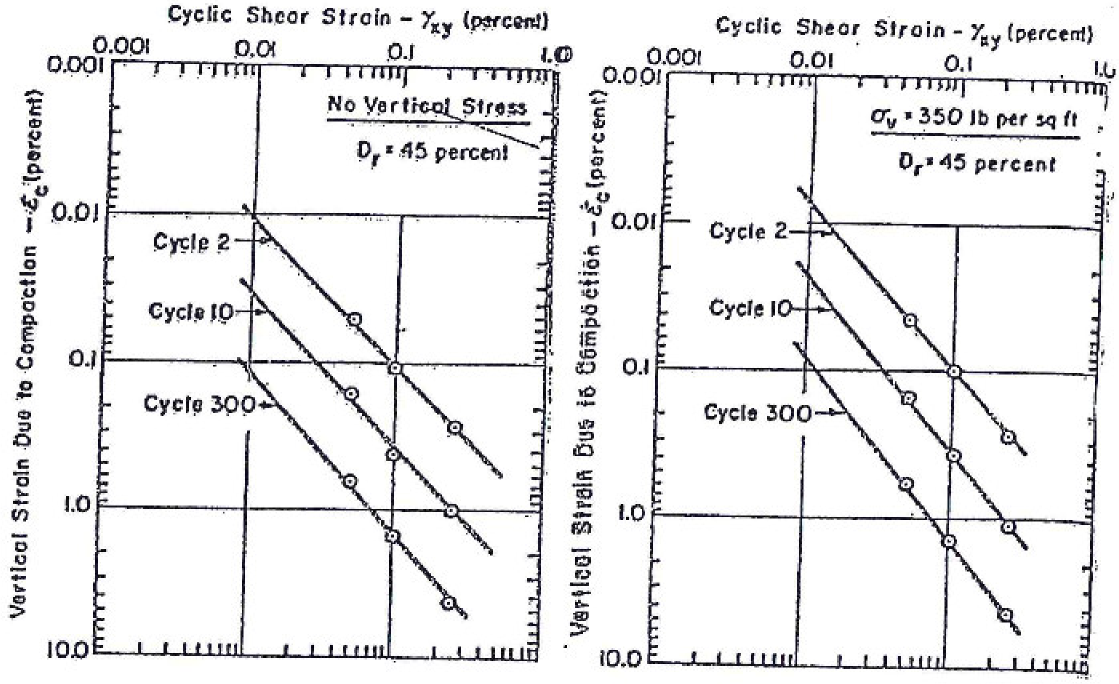

FIG. 17—VERTICAL SETTLEMENT—SHEAR STRAIN RELATIONSHIP FOR SILICA SAND (D_r = 45 %) (21)

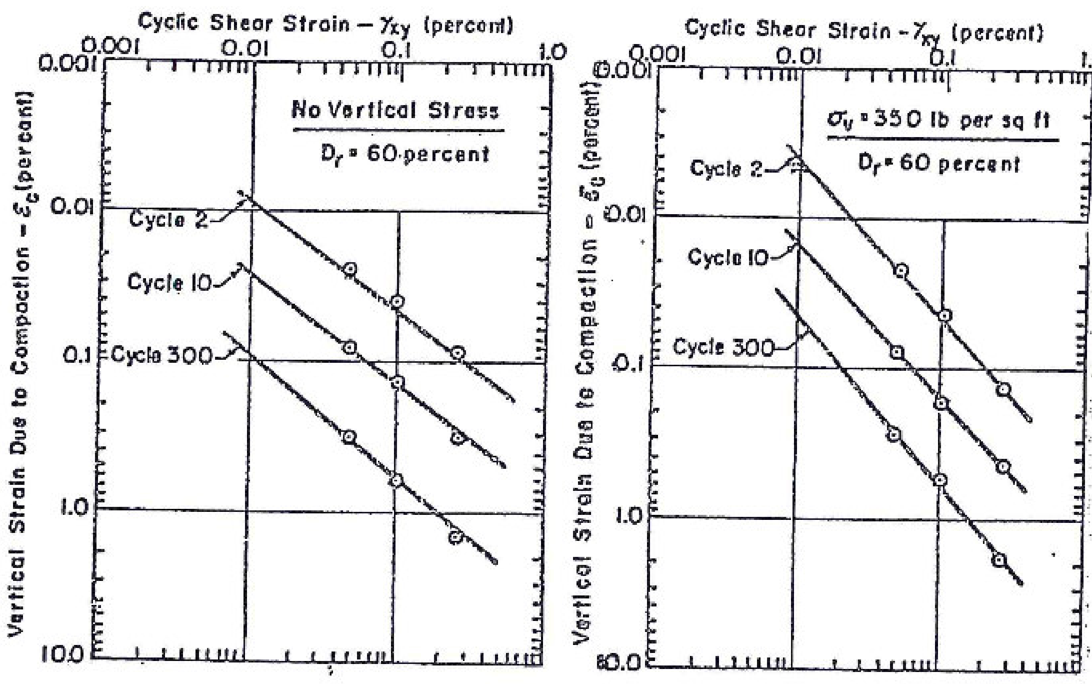

FIG. 18—VERTICAL SETTLEMENT—SHEAR STRAIN RELATIONSHIP FOR SILICA SAND (D_r = 60 %) (21)

CONCLUSIONS

It may be concluded from the above that the effect of earthquake loadings on soil depends upon (a) the soil type and its conditions in situ and (b) the earthquake ground motion as defined by the significant acceleration amplitude and number of cycles.

The behaviour of both cohesive and granular soils subjected to cyclic loadings such as earthquakes can be predicted with a degree of accuracy sufficient to allow predicitons to be made for design field conditions. However, before the above procedures and methods can be used with any degree of confidence, earthquake ground motions at various locations in the Caribbean must be defined. There is evidence that our region is subjected to more distant earthquakes of large magnitude than California. These are likely to produce longer period ground motions and care must be taken in making any assumptions as to likely ground motions.

In the future, data on local earthquake strong ground motions should be available from the Seismic Research Unit of the University of the West Indies since they are embarking on a programme of field installation of strong ground motion instruments at various locations in the Caribbean. In addition, the Seismic Research Unit has access to the SHAKE 3 computer programme developed at the University of California. This computer programme provides an analytical solution defining the anticipated ground motion spectrum for a given site once the base rock motion is assumed. Until such time as a sufficient number of strong ground motion instrument readings become available, this computer programme may provide the necessary ground motion input for predicting insitu soil behaviour.

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