THE UNIVERSITY OF MICHIGAN

INDUSTRY PROGRAM OF THE COLLEGE OF ENGINEERING

GENERAL REPORT ON SESSION II - EFFECTS ON SOILS AND ROCKS

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INTRODUCTION

This session is concerned with the behavior of soil and rock materials under abrupt, infrequent, and transient loading including impact and non-linear vibration processes involving large amplitudes. These loadings can cause heavy damage to the soil structure with resulting damage to any building or facility resting thereon. When applied to metals, the behavior under this type of loading is often designated as "low-cycle fatigue".

The topics included in Session I indicate some of the loading conditions which may cause damage to soils. Blast loadings either from nuclear explosions or from commercial blasting processes will cause primarily a single large amplitude of loading with possibly some reverberations of lesser importance. Damage is generally induced in the first loading, but multiple blasts will cause additional increments of damage. Earthquake motions may introduce from 1 to 100 cycles of motion of sufficient amplitude to cause significant damage. In certain earth handling procedures, controlled failures are desired and are produced by repeated loadings from vibratory compactors or repeated impact machines. The objective is to preload the material during construction to such a condition that subsequent loadings will cause no further changes in the soil structure. Finally, repeated loadings may be applied intermittently and at a rate which produces effects similar to repeated static loadings. Problems associated with land locomotion, or with the maintenance of unpaved airfields could fit into this category.

After the type of loading has been determined it is necessary to define "failure". A sample of soil loaded in the unconfined or triaxial test condition may rupture or merely deform excessively, depending upon the characteristics of the soil and the testing conditions. If the loading is of the controlled load type, the sample will deform continuously at failure because the load continues to act throughout the deformation. In the controlled deformation test, load is developed by the resistance of the sample to the rate of applied deformation. Under this test condition it is possible for the load (or stress) to reduce during the test with the result that the stress available at a limiting deformation is often designated as the failure stress. In the unconfined and triaxial tests, the strength of the sample is primarily dependent on its resistance to shearing stresses. However, when the sample is laterally confined and subjected to vertical loading, the deformation is primarily governed by the resistance of the sample to changes in volume.

The resistance of soil to applied loads depends on the combined resistance of the structural frame composed of the solid particles and the resistance of the pore fluid. If the pore fluid consists of water and gas (usually air), then the resistance of this mixture to a change of volume is low and the resistance of the soil structure predominates. When the soil is saturated, the resistance of water to volumetric compression is many times greater (i.e., the bulk modulus of elasticity, $E_v = 300{,}000$ psi for water) than that of the soil skeleton, and the soil mass is nearly incompressible with respect to changes of volume. However, the soil skeleton still must provide the resistance to shearing deformations.

The shearing strength of a soil is usually described by the Mohr-Coulomb equation, based upon effective stress parameters, as

$$\tau_f = c' + (\sigma - u) \tan \phi' \tag{1}$$

in which c' is the effective cohesion intercept,

σ is the total normal stress on the failure plane,

u is the total pore pressure,

and ϕ' is the effective angle of internal friction.

Under dynamic loading conditions, Equation 1 is still appropriate for describing failure conditions, but it is necessary to include the effects of dynamic pore pressures and the stresses which result from particle motions. If we consider first a unit volume of dry cohesionless soil at a depth z below the horizontal surface, the static vertical pressure on this volume is caused by the overburden, and is described as

$$\sigma_v = \gamma z \tag{2}$$

Now, if this column of soil of height z above the reference point is temporarily considered as a rigid body, and is set into vertical oscillation with an acceleration

$$a = A\,\omega^2 \sin\omega t \tag{3}$$

the vertical stress on the horizontal plane varies with time as

$$\sigma_v = \gamma z + \frac{\gamma z a}{g} = \gamma z\left(1 + \frac{a}{g}\right) \tag{4}$$

Thus, when the vertical acceleration of this column has an upward acceleration equal to the acceleration of gravity, or $a = -1.0\ g$, there is no effective vertical stress on the horizontal plane. This example is extremely crude, but is presented to point out that vertical accelerations of particles within a soil mass cause a variation in the normal stress, σ , and a resulting change in the shearing stress, τ [from Equation 1], which act on horizontal planes.

A direct effect of the transient pore pressures developed during dynamic loadings is to produce a change in the term, u , in Equation 1 which has a direct influence on the shear strength of the soil, on all planes through a given point. Because the development of pore pressures throughout a soil mass is seldom uniform, as a result of dynamic loads, transient hydraulic gradients are often initiated. The vertical component of this transient hydraulic gradient is described by

$$\Delta i_z = \frac{\partial h}{\partial z} \tag{5}$$

and the transient pressure gradient is

$$\Delta i_{p_z} = \gamma_\omega \frac{\partial h}{\partial z} = \frac{\partial u}{\partial z} \tag{6}$$

The upward transient pressure gradient acts in a manner similar to the upward acceleration forces and both tend to decrease the intergranular stresses.

This discussion of the effects of soil particle accelerations and transient pore pressures was included to emphasize the effects the stress and displacement environment has on the shearing strength of soils. These effects must be evaluated continuously, in addition to the consideration of phenomena which have previously been found to be of importance in the study of low-cycle fatigue of metals.

LOW-CYCLE FATIGUE OF METALS

Before considering the effects of high-amplitude repeated stresses on the behavior of soils it is useful to review some of the findings from the low-cycle fatigue behavior of metals. The range of cycles usually considered is from 1/4 cycle (static loading or dynamic impact tension leading) to 10^4 or 10^5 cycles. Nearly all of the specimens have been subjected to bending or tension stresses and failure is defined either as complete rupture of the specimen or detection of the first fatigue crack. For small specimens the number of cycles between detection of the crack and failure is usually small and complete rupture is often selected as the criterion for failure. Testing conditions are either constant load, constant stress, or constant deformation, repeated each cycle until failure occurs. When the test information is plotted in the usual manner, that is, using nominal stress as ordinate vs. logarithm of the number of cycles as abscissa, the test curve (which really represents a scatter band of test information) is generally quite flat in the region of low cycles. On the other hand, if the true strain at first maximum load is plotted against the number of cycles to failure, the relation is fairly well represented by a straight line on the semi-log plot, as illustrated in Figure 1.

It has also been demonstrated in low-cycle fatigue as well as for the usual fatigue situation that the range of stress available is a function of the mean stress applied. The terms which are described on Figure 2 are important for understanding subsequent discussions. The curve illustrated on Figure 2a is a sinusoidal variation of stress about some mean value, designated as the steady stress, σ_s . With one set of notation we can denote the steady stress by σ_s , and to this we can superpose an alternating stress σ_a . Another way of describing the same

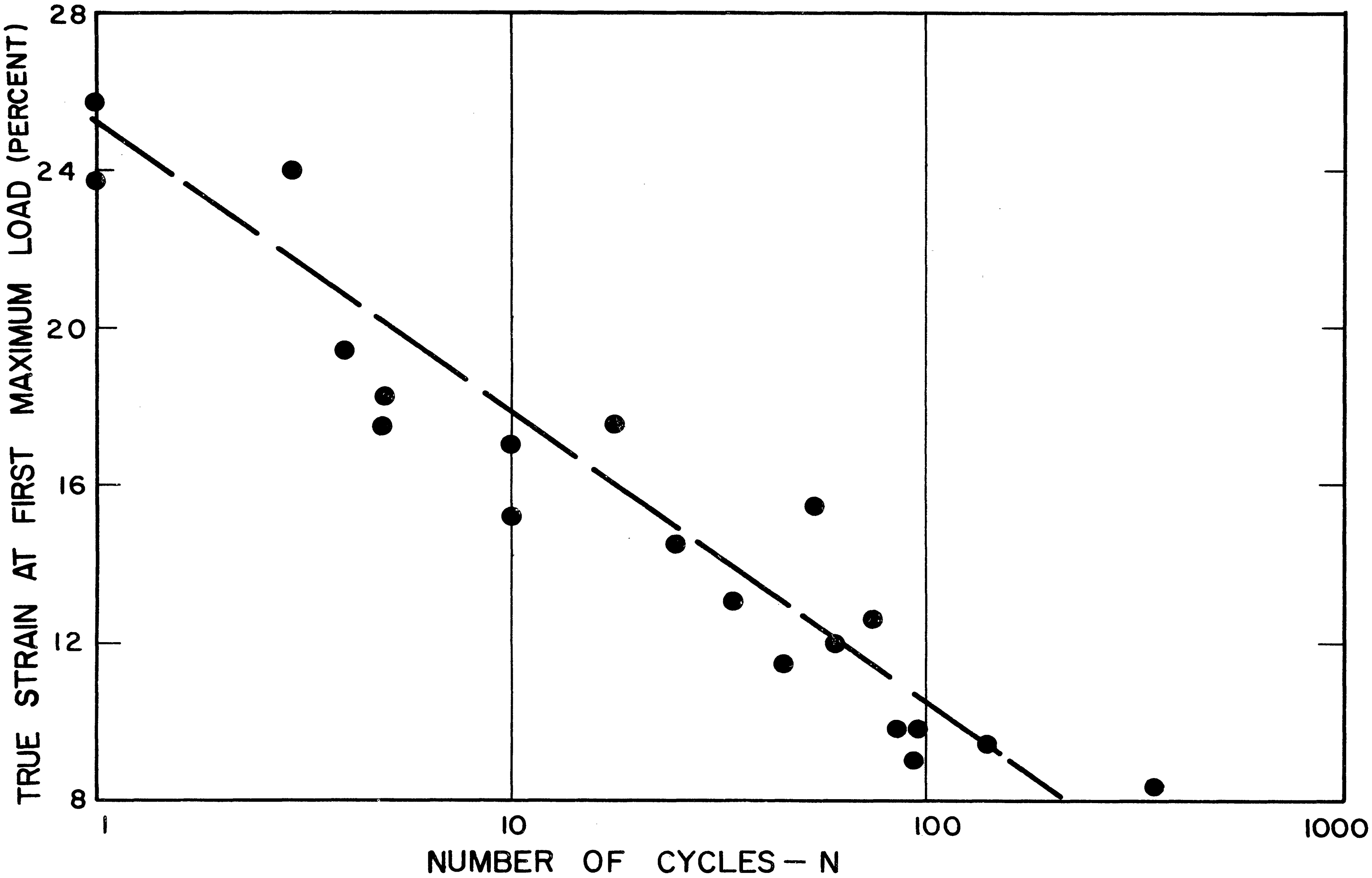
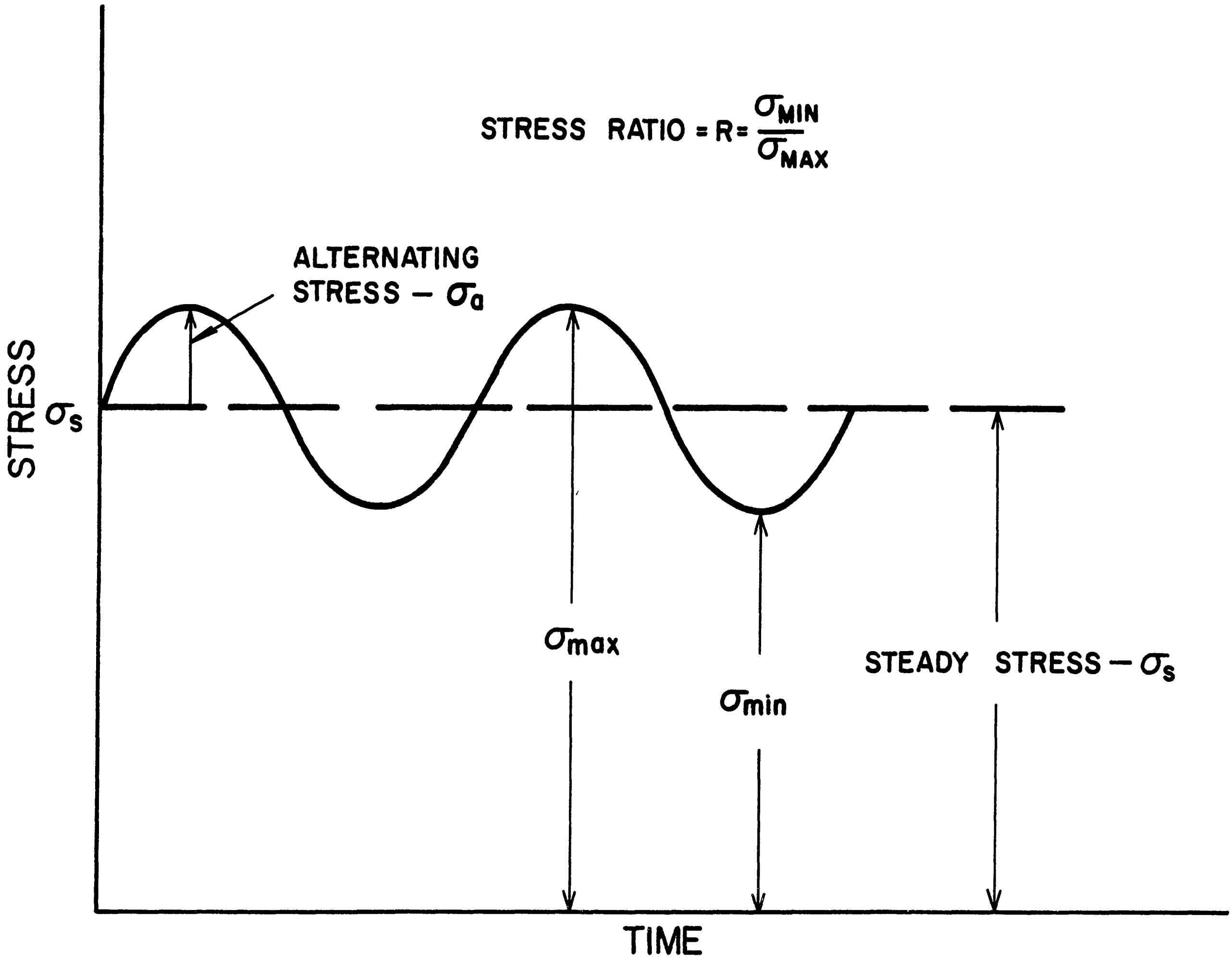
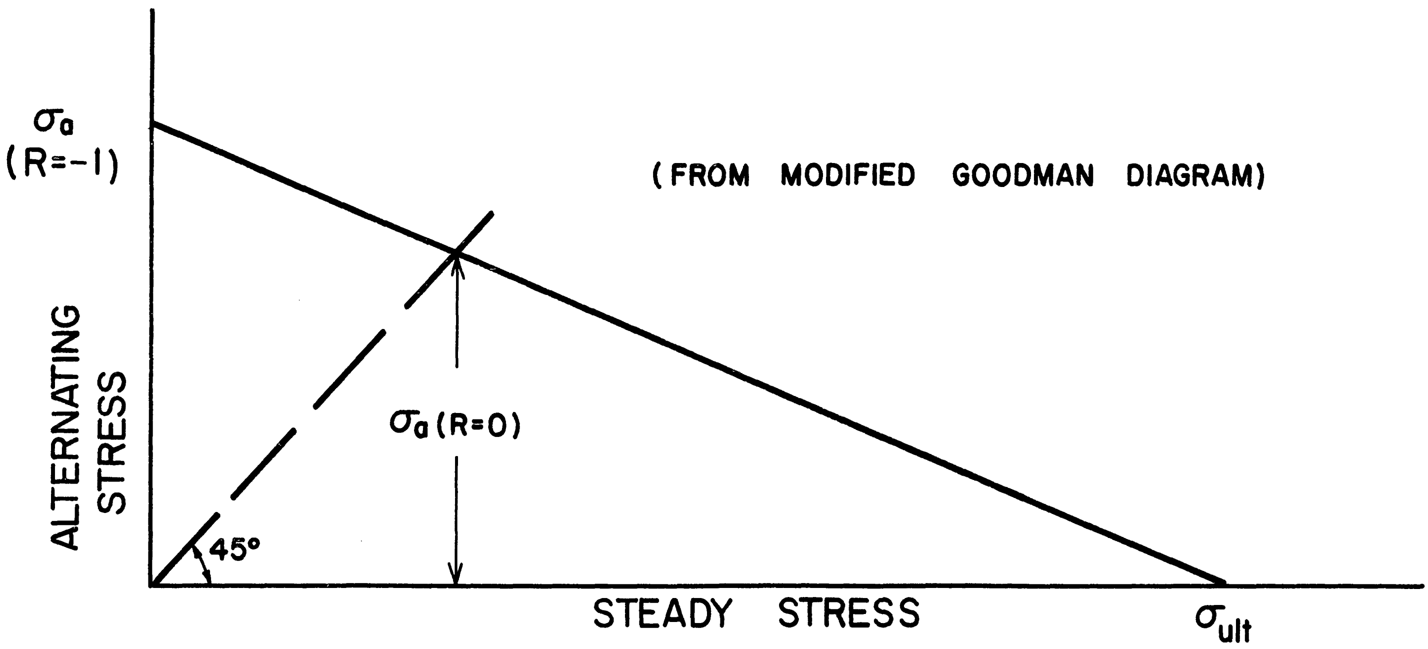


Figure 1. True Strain at First Maximum Load vs. Fatigue Life. - 0 to Tension Tests of Steel Specimens (From Yao and Munse, 1963).



(a)



(b)

Figure 2. Influence of Range of Stress on Fatigue Strength.

phenomena is to use the maximum, σ_{max}, and the minimum stress, σ_{min}, in the loading cycle. This latter method is often described in terms of a stress ratio, R, which is the ratio of minimum stress to the maximum stress in the loading cycle. Using this terminology, stresses varying from zero to a maximum have $R = 0$, and completely reversed stresses from tension to an equal value in compression have $R = -1$.

Figure 2b illustrates the effect of the steady stress on the allowable range of stress in the fatigue of metals. This way of presenting fatigue data was described by Soderberg (1933), and is slightly easier to use than the modified Goodman diagram which was devised earlier. The effect of steady stress on the allowable range of alternating stress was noted by Wöhler (1858) during his classical experiments of fatigue of metals. This diagram illustrates the reduction in allowable alternating stress as the steady stress is increased, but in addition it shows that the maximum stress in the loading cycle may be increased if the minimum stress increases also as

$$\sigma_{min} = \sigma_{max} - 2\sigma_{allow}$$

A comprehensive study of low-cycle fatigue of metals was undertaken by Yao and Munse (1962, 1963). They included a review of 69 references on the subject and described their own test results and evaluation of other test results. Conclusions reached from this study indicated that (a) there is presently no general analysis applicable to all low-cycle fatigue test conditions, (b) the shape of the load-time curve is an important factor in analyzing low-cycle fatigue tests, (c) the extent of the time effect on low-cycle fatigue behavior, particularly with respect to creep and crack propagation still remains to be explored, (d) the use of strain rather than stress is more desirable in low-cycle fatigue studies because of the plastic deformation that takes place during such tests, and (e) the fatigue hypotheses based on strain, although developed from limited data, exhibit good agreement with the test results.

In addition to the relation between true strain at first maximum load vs. log of number of cycles being indicated as a straight line for various metals, it has also been indicated that the cyclic tensile change in plastic strain, expressed as per cent, can be used with the number of cycles to give a linear diagram of failure on a log-log plot, that is, log of cyclic tensile change in plastic strain vs. log of number of cycles. Test data are available which support this concept.

Only a limited number of tests on low-cycle fatigue of metals loaded in compression have been conducted. Tests by Newmark, Mosborg, Munse, and Elling (1951) on gray cast iron specimens have indicated that fracture occurred under repeated compressive loadings in the same general manner as under static loadings. The fractures were of a diagonal shear type but the specimens which failed in fatigue were generally more broken up than the ones that were tested statically.

The important quantities which affect the low-cycle fatigue strength of metals then include the type of testing (stress controlled or strain controlled), the range of stress or strain applied, the stress ratio, the shape of the load-time curve, particularly the "hold time" at the maximum stress, and changes in material properties throughout the tests (some materials were found to strain-soften at high values of mean strain while for lower values of mean strain occasionally strain-hardening occurred).

Finally, several series of tests have indicated that fractures of metals resulting from low-cycle fatigue loadings may show deformations ranging from those corresponding to ordinary long-life fatigue failures to those obtained in a static tensile failure, thus significant deformations may occur under low-cycle fatigue tests.

The general influences of the different variables on the low-cycle fatigue of metals are useful as guides for the prediction of what may occur in tests of soils under high amplitude repeated loadings. However, it must be recognized that the failure condition of metals is quite different than that which occurs in soils. Under fatigue loading conditions, metals behave more nearly as a continuum and a fatigue failure is a local progressive failure exhibited by the propagation of a crack or cracks through a zone stressed in tension. Soils, on the other hand, behave more nearly as particulate materials having a mechanical response which is strongly dependent upon the local stress conditions and previous stress history.

BEHAVIOR OF COHESIONLESS SOILS

Compaction Resulting from Dynamic Loads.

The behavior of cohesionless soils under dynamic loads depends upon a number of factors. In a cohesionless material the term c' in Equation 1 is equal to zero. Consequently, the shearing strength depends upon the effective normal stress on the failure plane and the effective angle of internal friction under the given loading condition. The effective angle of internal friction depends upon the grain size, shape, roughness, and the void ratio of the material as well as the degree of saturation and the rate of loading.

Terzaghi (1925) pointed out one influence of dynamic loading of cohesionless soils established by an upward flow of water through the soil. When this flow produced a hydraulic gradient of about 1.0 the sand grains essentially floated and the permeability increased markedly. At the same time the shearing stress had reduced to zero and the sand was in a "quick" condition. Whenever a mass of loose saturated sand is disturbed by impact or vibrations the sand particles tend to fall into a more compact position. During the time it takes a sand grain to move to its new position it is temporarily partially supported by the pore fluid and the sand tends to become "quick". This has often resulted in flow slides whereby embankments of submerged sand run out almost horizontally as a fluid. Casagrande (1936) described this phenomena and proposed a term called the "critical density" described as the density (or void ratio) at which continuous shearing deformations may occur without change of volume. At a lower density (higher void ratio) the sand structure reduces in volume during shearing stresses and temporarily transfers its weight to the pore fluid. At greater than the critical density, shearing deformations produce expansion of the grain structure, which tends to produce tension in the pore fluid. This led to a widespread acceptance of the idea that a particular sand had a unique value of critical density (or critical void ratio). Further studies of the volumetric expansion or contraction of sand during shearing deformations were carried out by Chen (1948) and Geuze (1948), who showed that the critical states of density or void ratio as defined by no volume change during small amounts of shear were functions of the confining pressure and methods of testing. Thus there is not a specific "critical void ratio" for a given sand. For saturated sands the probability of compaction as a consequence of impact or repeated loads is primarily a function of the initial void ratio, the confining pressure, and the intensity of the dynamic loading.

Because of the susceptibility of loose sands to compaction by the introduction of dynamic loadings, it was anticipated that controlled blasting by small charges of dynamite or other high explosives should be an effective method of providing compaction. The study by Lyman (1942) showed that blasting was an effective way to compact loose sands but in order to be successful it was necessary to run field tests at each site to establish the proper charge and spacing. A further description of this method was given by Prugh (1963). The probability of developing an increase in density by blasting depends upon the ability of the material to fall back into a more dense position after it has been disturbed by the charge. Consequently, this method is relatively inefficient close to the surface of submerged slopes where the sand tends to roll back into position down the slope rather than being confined as it falls. Controlled blasting has been used, however, in determining whether submarine slopes might be susceptible to flow slides caused by earthquakes (Kummeneje and Eide, 1961).

An interesting study of the liquefaction and compaction of granular soils by blasting has been reported by Florin and Ivanov (1961). They found that in impulse tests, loose saturated sands were readily converted into a completely liquefied state over the entire depth of the test stratum. At this time the soil had a bearing resistance which approached zero. Then, with a time rate which depended on the permeability of the material, the sand settled back into place, beginning at the bottom, with the upper layer remaining in the liquefied condition for a longer time. The ability of the sand layer to be liquefied depends upon its initial void ratio, the range of confining pressure and, of course, the amount of energy introduced by the blast. The authors concluded as a result of many field observations, that loose sand can hardly be liquefied at depths ranging between about 10 to 15 meters below ground level. This indicates that surcharge with any material can be used as a method for reducing sand liquefaction. Florin and Ivanov also produced liquefaction of the sand by steady-state vibrations. By this method liquefaction was not caused simultaneously throughout the depth of the test stratum, but the first vibrations liquefied the upper layer which carried a comparatively small confining pressure. This then reduced the pressure of the overburden on the lower layers and as a result the latter passed into the liquid state with continued vibration. Thus, the zone of liquefaction extended downward. They also indicated that all sufficiently loose cohesionless soils of any grain size may be liquefied, but the time required for compaction depends upon the permeability. Coarse grained soils compact very rapidly.

Because of this susceptibility of loose saturated sands to compact and possibly to liquefy as a result of impact or repeated loadings, it is probable that difficulties will be encountered with this type of material when it is supporting structures under earthquake loadings. Numerous investigations have ascertained that the structural damage resulting from earthquakes has been a result of liquefaction and compaction of loose saturated sands, for example in the Chilean earthquake of May 1960 (Steinbrugge and Clough, 1960), the Coatzacoalcos, Mexico earthquake of August 26, 1959, (Diaz de Cossio, 1960), the Jaltipan earthquake, Mexico (Marsal, 1961), the Anchorage, Alaska earthquake, of 1964 (Shannon and Wilson, 1964), and the Niigata earthquake, Japan, 1964 (Falconer, 1964, and the January 1966, Vol. 6, No. 1 "Soil and Foundation" issue of the Japanese Society of Soil Mechanics and Foundation Engineering). Extensive soil investigations have been conducted following the Anchorage, Alaska, earthquake (Shannon and Wilson, 1964) and the Niigata, Japan earthquake.

In order to interpret some of the results from the Anchorage, Alaska, earthquake, Seed and Lee (1964) ran pulsating load tests on samples of fine silty sand. Specimens were tested in a triaxial device in which the vertical stress was alternated and the lateral stress was maintained constant. The frequency of load pulsation was 2 cycles per second and the pore pressure response and the change in length of the sample were recorded throughout the test. The triaxial test set-up with confining pressure indicated as σ_3 and the pulsating axial stress indicated as $\Delta\sigma_1$ is shown on Figure 3. In one series of tests the axial stress was always maintained greater than the lateral confining pressure, to maintain the maximum principal stress in the vertical direction. In a second series of tests it was possible to produce a tensile load on the upper surface of the specimen to reduce the axial stress to less than the confining pressure. For these tests, which were described as stress reversal tests, the maximum principal stress alternated from the vertical to the horizontal directions thereby reversing the direction of the maximum shearing stresses. Figure 3 indicates typical test results illustrating the pulsating principal stress difference plotted as ordinate and the logarithm of number of cycles to cause failure plotted as abscissa. This diagram is quite similar to the one obtained for metals in the low-cycle fatigue range. Failure of the triaxial samples was either by complete collapse after small initial axial deformation or by a more gradual deformation with number of cycles. In the latter case an axial

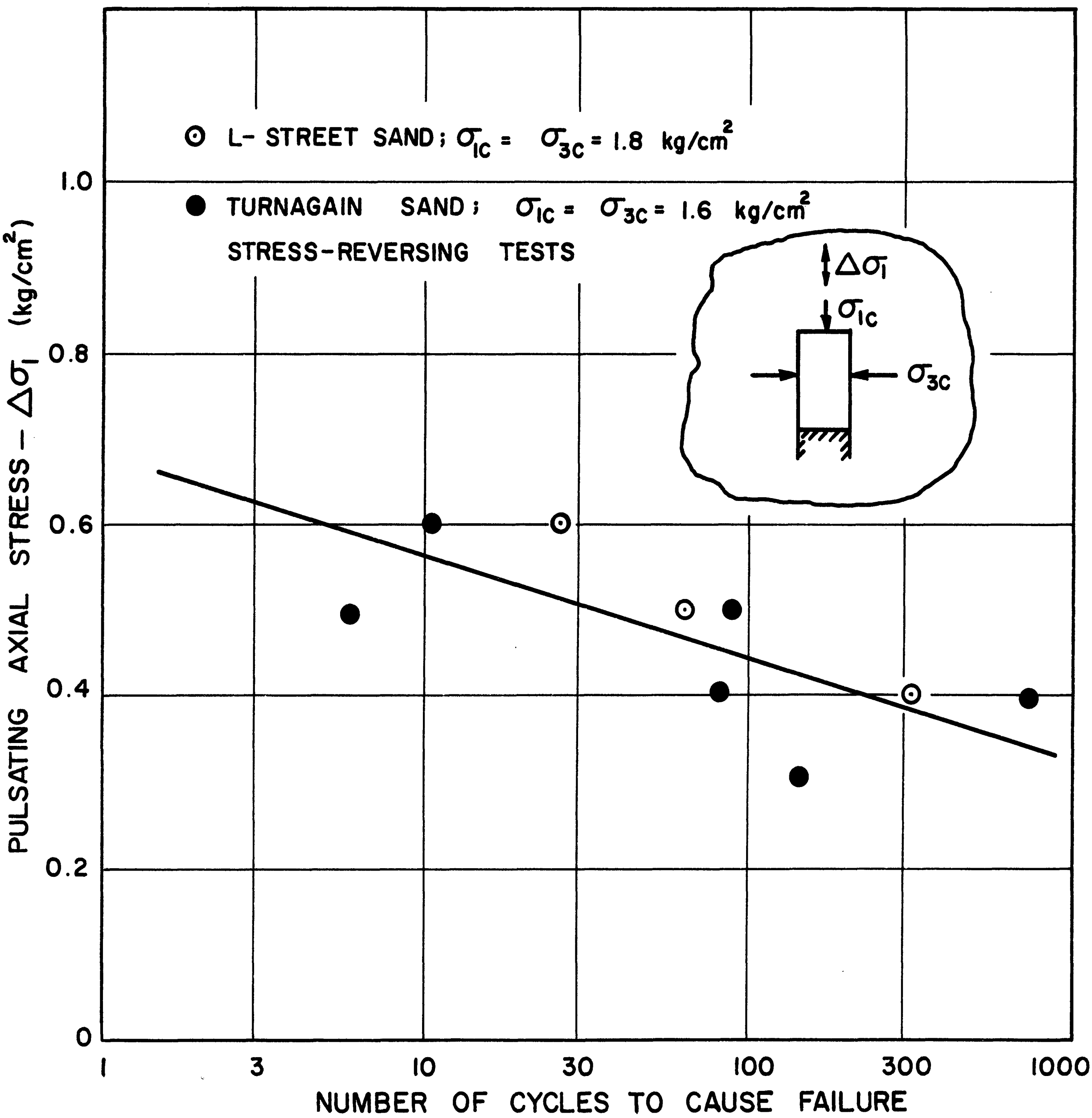


Figure 3. Pulsating Axial Stress vs. Number of Cycles to Failure for Triaxial Samples (From Seed and Lee, 1964).

strain of 20 per cent was taken as the failure condition. The effects of anistropic consolidation and the range of stress were found to be significant.

When evaluating triaxial tests of cohesionless soils, it is necessary to determine the influence of the testing conditions throughout the test. One convenient method of plotting triaxial test data is shown on Figure 4 where the ordinate represents the vertical or axial principal stress σ_1 , and the abscissa is a function of the lateral confining pressure $(\sqrt{2}\ \sigma_2 = \sqrt{2}\ \sigma_3)$. This diagram represents a plane in the stress space that includes the σ_1 axis and the hydrostatic axis OC , for which $\sigma_1 = \sigma_2 = \sigma_3$. The Mohr-Coulomb failure conditions for a cohesionless material are represented by the dashed lines OA and OB . These failure lines indicate the stress conditions which relate to excessive strain or collapse of the specimen. A point (F) on the failure envelope may be reached along several stress paths, for example, the one normally followed in the triaxial test includes a hydrostatic stress $\sigma_o = \sigma_3$, which occurs along OC_1 in Figure 4, then an increase in axial loading, C_1F , without change in the cell pressure. Point F could also be reached by path OC_2F which involves a hydrostatic stress σ_o , and a shearing stress τ_o on the octahedral plane. It should also be possible to describe on this diagram the limiting stresses for failure under a given number of cycles of repeated loadings. The line ON on Figure 4 is an indication of how repeated loading information could be included on this type of diagram.

At a lower level of energy input is the category of forced compaction of soil by vibratory or impact devices. Steuerman (1939) described the process of vibroflotation in which a vibrating probe is inserted

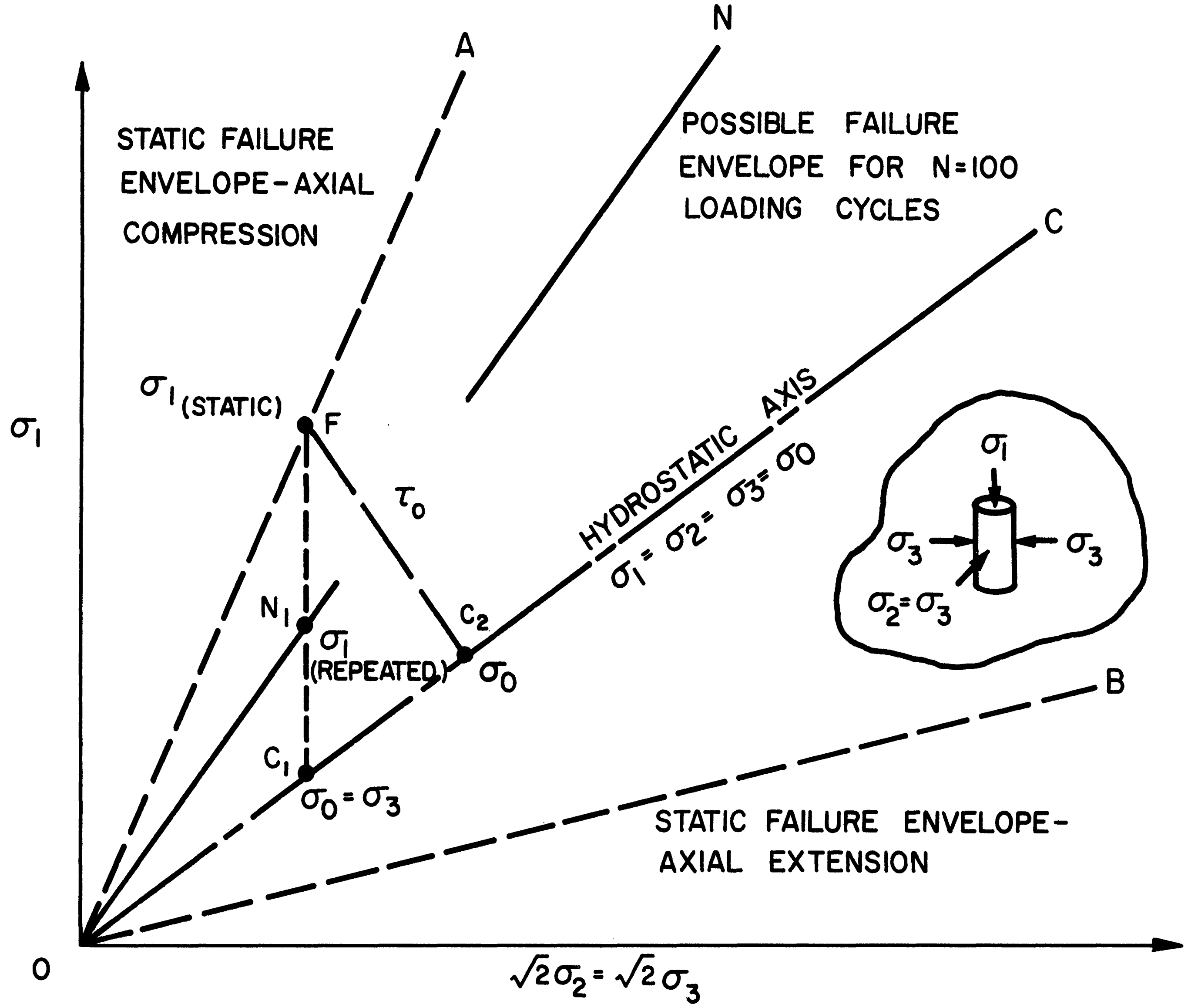


Figure 4. Representation of Mohr-Coulomb Failure Conditions for the Static and Repeated Loading Triaxial Tests of Cohesionless Soils.

vertically into sand layers to compact the loose sand around it. Horizontal impacts are applied to the sand layer by this device. D'Appolonia (1953) has presented a very interesting study on the use of vibroflotation and the evaluation of its effectiveness through the use of the relative density concept. He has indicated an influence pattern for determining the compaction efficiency and for establishing the horizontal spacings of the vibroflot operations. This influence pattern is a measure of the damage induced into the soil by vibrating impacts. A further method of compacting granular materials by repeated or vibratory means is through the use of a vibrating roller which is moved along the surface and imparts energy downward into the sand layer. The compaction at a particular depth depends upon the initial void ratio of the material, its degree of saturation, the intensity of the surface vibratory loading, its speed of travel over the surface, and the number of times the vibrator passes over a particular spot. D'Appolonia (1966) has indicated the effectiveness of this method in compacting loose dune sands to a depth of 25' by this method. He has incorporated the concept of "energy ratio", $\left(\frac{\text{acceleration}}{\text{frequency}}\right)^2$ as introduced by Crandell (1949), and the relative density, to give relations of strain ratio which permit estimates of the surface settlements of sand layers by high amplitude vibratory loads.

Stress-Strain Behavior of Cohesionless Material Under Dynamic Loads

Numerous investigations have also been made on the behavior of cohesionless materials under rapid loadings which cause ultimate failure of the sample. Casagrande and Shannon (1948) presented test results of triaxial tests of dry Manchester sand loaded by a falling beam type of apparatus. They found an increase in shearing strength of 10 to 15 per cent under the dynamic load. Seed and Lundgren (1954) investigated the strength and deformation characteristics of saturated specimens of a fine and a coarse sand under dynamic loadings. The rapid loading was applied by a falling weight impact testing machine. They found increases in strength of 15 to 20 per cent and about 30 per cent increase in modulus of deformation under dynamic loading above the values obtained from static loads. They also noted that dilatancy effects and lack of drainage contributed to the strength under dynamic loads. Taylor and Whitman (1954) reported the results of triaxial tests of saturated sands in which one of the major objectives was to measure the time change of pore pressures. This measurement of transient pore pressures has now been perfected to the point where they feel that rapid triaxial tests may be conducted with loadings in the order of 0.01 sec. with satisfactory measurements of pore pressures. Nash and Dixon (1961) developed a dynamic pore pressure measuring device with which they successfully recorded pore pressure changes in triaxial tests of saturated sands under strain rates up to 8,000 per cent per minute or about 220 inches per minute loading rate. The rapid loadings used in the tests by Nash and Dixon were obtained by loading a lever with a falling weight. As a result, this approximated a constant rate of strain type of test. In the course of loading the specimen to its maximum (loading time approximately 0.1 sec.) they found several short time drops in the stress level and simultaneous increases in the pore pressure. (See Figure 5). This indicates that the structure collapsed intermittently at a rate which was faster than the loading rate, but the change in structure permitted the sample to carry increasing axial loads until the maximum was reached. The time rate of these steps of structural collapse or grain readjustments should be of considerable importance in evaluating the behavior of sands under high rates of loading.

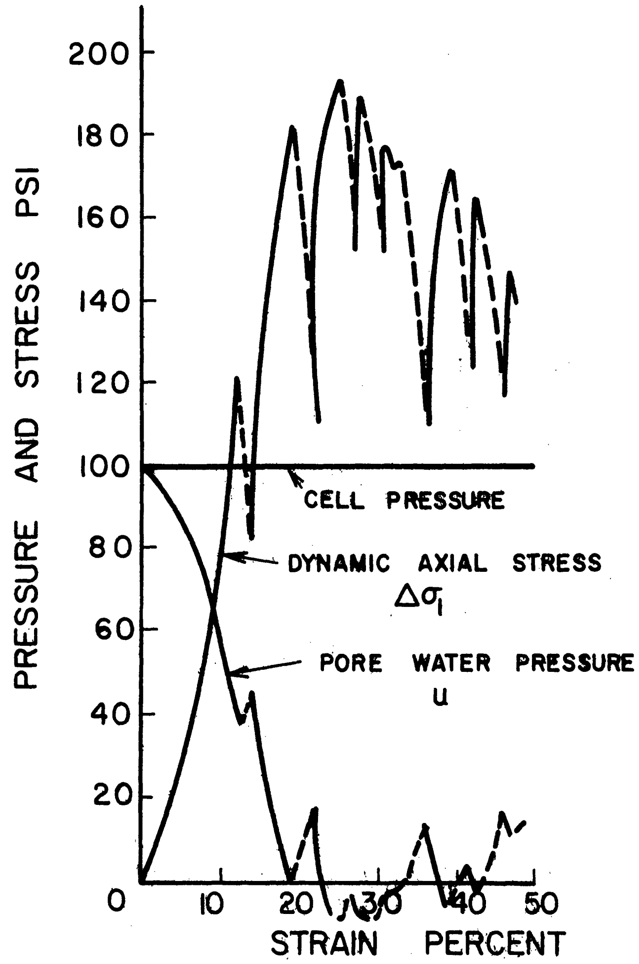


Figure 5. Dynamic Triaxial Test Results from Saturated Sand, Showing Sudden Collapses During Shear (From Nash and Dixon, 1961).

These foregoing tests of dynamic loadings on sand samples in the triaxial machines provided an undetermined lateral restraint of the sample. Generally the sand samples failed and the stress-strain curve was of the strain-softening type, Figure 6, curve A. When the sand is laterally confined the stress-strain curve is of the strain-hardening type similar to curve C, Figure 6 (after a small initial strain-softening portion). Thus the stress-strain behavior of sands are extremely dependent upon the type of confinement, the type of loading, and the intensity of the loading. It should be noted that as the intensity of loading becomes high the grains tend to break and compaction occurs as a result of these fractures. It should also be recognized that the deformations of granular materials are essentially inelastic when longitudinal strains greater then the order of about 10^{-4} are encountered. This implies a loss of energy to the soil through hysteresis during each repeated loading cycle during a test or during a compaction process.

To summarize this section, there are two principal factors which need to be evaluated in determining the behavior of cohesionless soils under dynamic loads. The first is the compaction or reduction in volume of the soil structure as a result of impact or repeated loads. This will cause eventual settlement of the soil surface and during the process of compaction, if the sand is saturated, there may be a temporary loss of shear strength by liquefaction. The second property of the soil of importance during dynamic loads is the stress-strain relation of the soil mass throughout the range of deformation to be encountered. This is a function of the grain characteristics and unit weight of the mass in its initial condition, the confining pressures, and the type, intensity, and duration of the dynamic

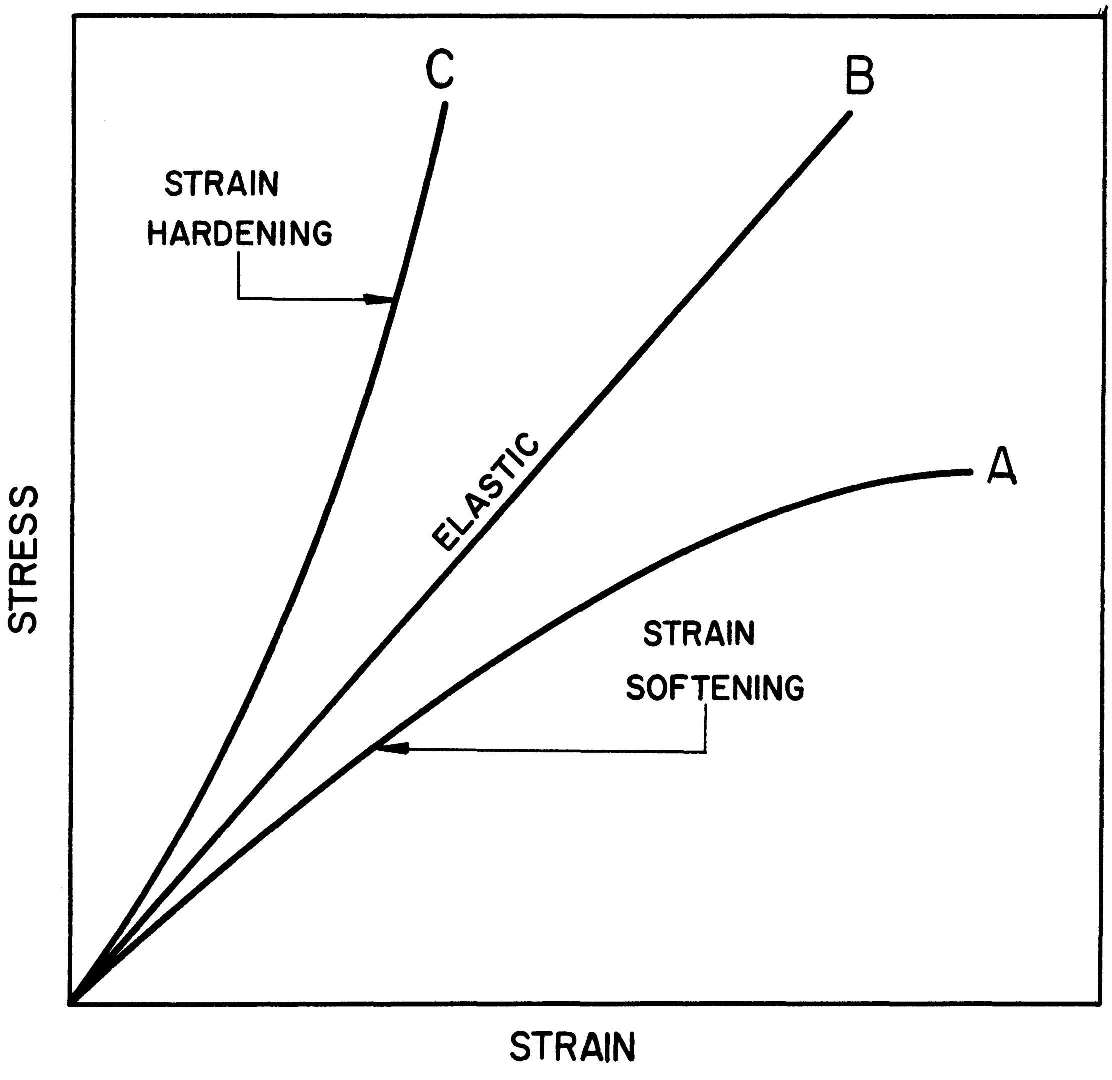


Figure 6. Types of Stress-Strain Curves.

loadings. The stress-strain characteristics govern the process of propagation of wave energy through a soil deposit and reflections of wave energy from discontinuities.

BEHAVIOR OF COHESIVE SOILS

Shearing Strength under Single Dynamic Loads

The term "dynamic load" is used here to denote loads which were applied much faster than in the conventional triaxial or unconfined test for which the time to apply the maximum load may take on the order of 5 to 30 minutes. The time to maximum dynamic loading is often on the order of 0.005 to 0.10 seconds, for example. A classic study of the behavior of cohesive soils under high rates of loading was made by Casagrande and Shannon (1948) on clays and soft rock. Their testing methods have been adopted and improved slightly by subsequent workers, but their test results are widely quoted and often compared with subsequent test results. Some of their more significant conclusions are listed below:

(1) The tests on Cambridge clay and Kaolin clay show that the water content has an important influence on the relation between time of loading and compressive strength. The per cent increase of the fast transient strength over the static compressive strength is greatest for specimens at the highest water content and least for specimens at the lowest water content.

(2) The fast transient compressive strength, taken at a time of loading of 0.02 sec., ranges between 1.4 and 2.6 times greater than the 10 minute static compressive strength for all tests on clays. For the same dynamic loading conditions the modulus of deformation (secant modulus determined at one-half the ultimate strength) for all types of clays tested

was found to be approximately twice that for the 10 minute static loading. Taylor and Whitman (1954) evaluated this strain-rate effect on ten different cohesive materials. Generally an increase in the strain rate from 0 to 1000 per cent per second increased the ultimate strength by a factor of 1.5 to about 3, which is of the same order of magnitude as that found by Casagrande and Shannon. Recent tests by Jackson and Hadala (1964) indicated that the strain-rate factor for the compacted clay used in their tests was of the order of 1.5 to 2.0. This strain rate factor was again the amount of increase in modulus of deformation and ultimate strength above that obtained for static conditions.

Thus the general conclusion can be obtained from these tests that the dynamic modulus and the ultimating shearing strength for cohesive soils will often be between about 1.5 and 2.5 times the static values.

Effect of Repeated Loadings

Because of the low permeability of cohesive soils and the relative insensitivity of the shearing strength to the confining pressure, it would be anticipated that the effects of transient hydraulic gradients and soil particle accelerations would be of considerably less importance for cohesive soils than they were for cohesionless soils. However, the influence of magnitude and rate of deformation, and periods of rest should produce significant changes and influences on the structure of a clay during successive loadings. Murayama and Hata (1957) subjected undisturbed samples of sensitive clay (sensitivity of about 8) to repeated deformations in simple shear at constant volume. For this material they found no significant effect of rates of shearing deformation between about 1/3 degree per second and 80 degrees per second, but they did find a distinct decrease in strength with number of stress applications from 1 to 200. They obtained straight-line plots of the data on a log-log presentation using degree of weakening as ordinate vs. number of repetitions as abscissa for three different angles of shearing deformation. In a discussion of this paper, Seed (1957) reported that repeated triaxial tests on compacted clays of low sensitivity did indicate an effect of speed of loading. He found that a frequency of application of 20 cycles per minute gave a distinct increase in deformation when compared with results from tests run at one application per two minutes. He attributed this change in deformation to thixotropic strength gains, which were particularly evident in comparing tests under continuous repetitions of loading with tests which permitted a rest of one day after each loading.

In order to simulate earthquake loadings of cohesive soils, Seed (1960), Seed and Chan (1964), and Seed and Chan (1966) have performed repeated loading triaxial tests with high-stress low-frequency pulses. In contrast to the frequency they used for estimating the behavior of highway subgrade materials, the rate of loading for the earthquake simulation tests were generally on the order of 1 or 2 cycles per second. Figure 7 illustrates the stress conditions which could act on soil elements which are subjected to the load pulsations produced by earthquakes. In Figures 7a, 7b, and 7c, the loading pulses represent the condition which might exist in an element of soil directly beneath a building footing as the footing pumps up and down when the building tends to rock because of horizontal impulses. In this element of soil there can be no tension loading, therefore all stress pulses must have zero stress as the lower limit. Consequently, the dynamic loading may be superposed on the static loading as indicated in

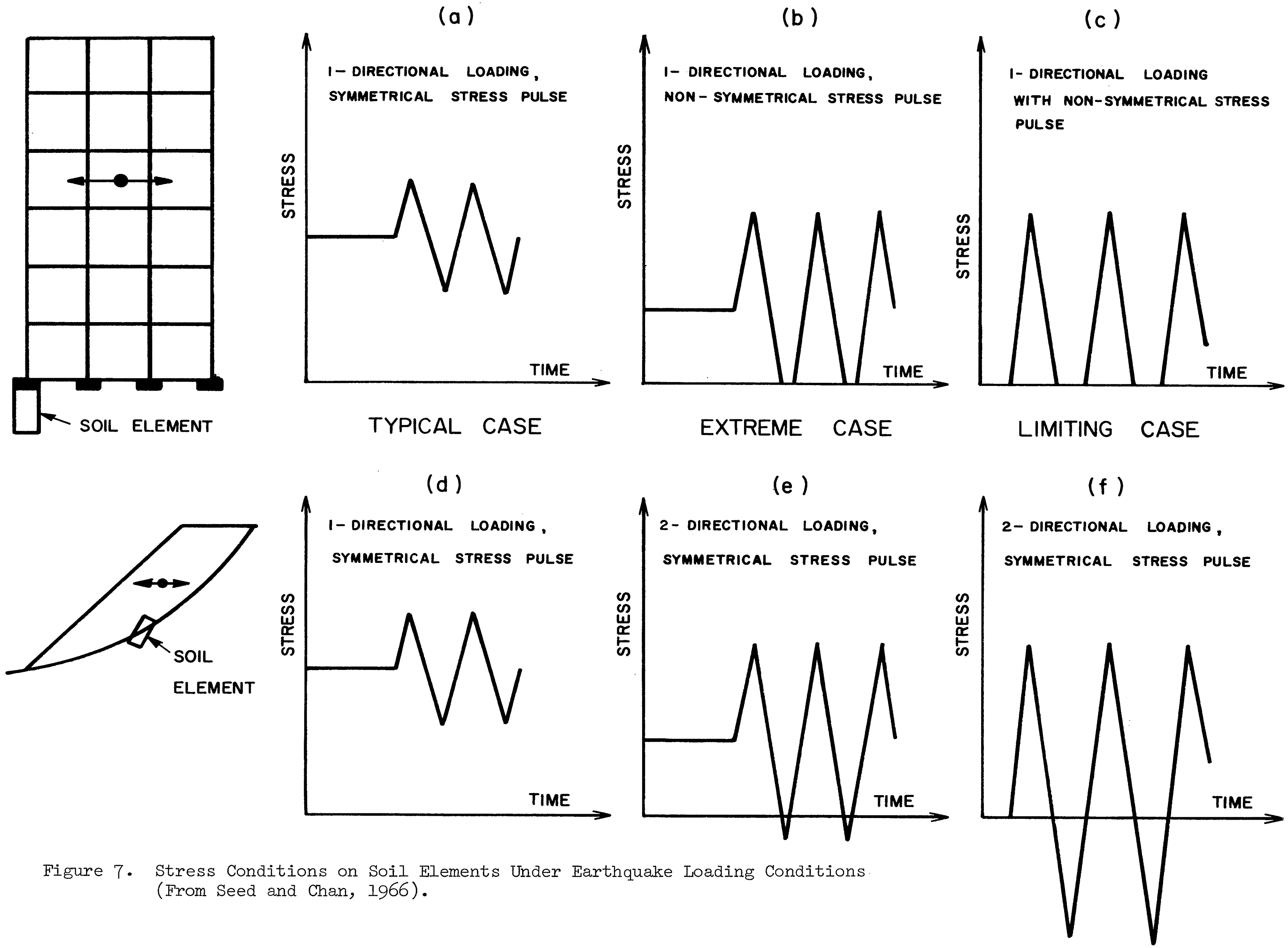


Figure 7. Stress Conditions on Soil Elements Under Earthquake Loading Conditions (From Seed and Chan, 1966).

Figure 7a or it may be represented as a series of isolated pulses as indicated in Figure 7c.

The other situation indicated by Figures 7d, 7e, and 7f represents several possibilities which may occur in a soil element within the zone of sliding of an earth embankment. The time-loading pattern may vary from a dynamic oscillation superimposed upon a steady stress or the dynamic loading may completely reverse its direction as indicated in Figure 7f. In the dynamic triaxial tests the confining or cell pressure was maintained constant and the axial stress was varied with time. If the axial stress at all times throughout the stress pulse was greater than the confining pressure then the direction of the maximum principal stress was always vertical, but if the vertical stress pulse reduced the axial stress to a value of less than the confining pressure, then the direction of the maximum prinicpal stress changed and it was termed a "stress reversal" type of test. Figure 8 indicates a type of test result obtained as plotted on the stress-strain diagram. With a confining pressure of 1 kg/cm^2 the limiting value of the principal stress difference $(\sigma_1 - \sigma_3)$ amounted to 3 kg/cm^2 while under the dynamic single impulse type of test, with a time of loading 0.2 sec., the ultimate strength was slightly greater then 4 kg/cm^2 . As indicated on Figure 8, if a factor of safety of 1.5 was utilized in a particular design the allowable $(\sigma_1 - \sigma_3)$ value would be 2 kg/cm^2 with an associated axial strain of approximately 5 per cent. After the confining pressure was applied and the principal stress difference value of 2.0 kg/cm^2 was added, then the pulsating axial stress of approximately $\pm$ 0.7 kg/cm^2 was applied to the specimen. The stress-strain path is indicated by the zig-zag lines on Figure 8, with the number at the right

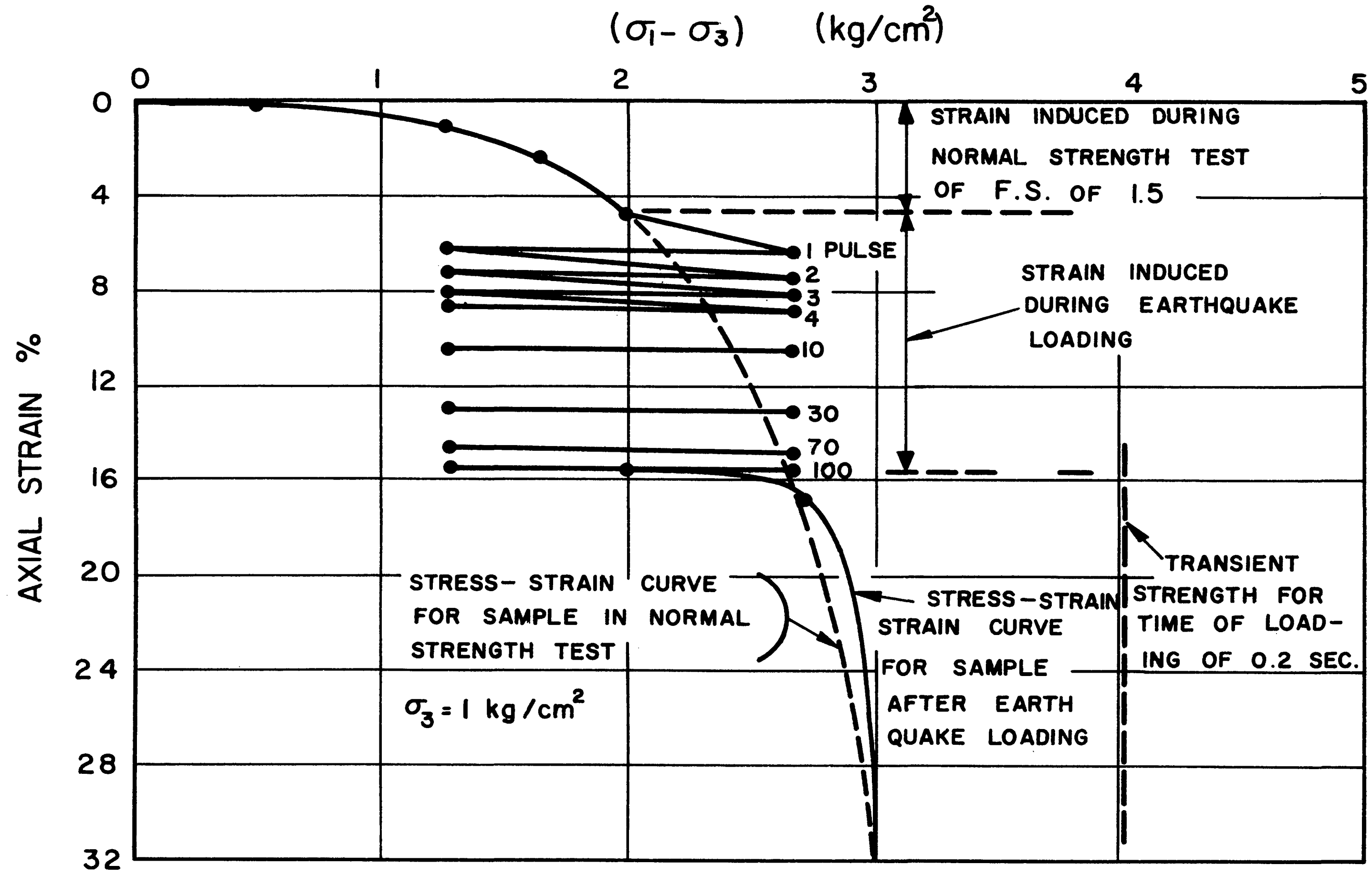


Figure 8. Stress vs. Strain Relationship in Simulated Earthquake Loading Test on Vicksburg Silty Clay (From Seed, 1960).

end of this zone indicating the number of pulses. After 100 pulses the axial strain was nearly 16 per cent and at this condition the stress-strain curve was continued under static loading as indicated. This particular diagram indicates that the total strain anticipated under the static plus 100 repetitions of the dynamic value of $(\sigma_1 - \sigma_3)$ produced a strain very close to that which would be anticipated from this total value of axial stress according to the static stress-strain curve.

Figure 9 illustrates the relation between the number of pulses and the axial strain for pulsating stresses on samples of silty clay. It is significant that the failure occurs so abruptly over the entire range of loading indicated. The circled numbers indicate the pulsating principal stress difference expressed as a per cent of the static compressive strength. These curves also show the influence of the pulse shape and frequency of application on the number of pulses to failure. As was indicated in previous studies of low-cycle fatigue of metals, the dwell time at the maximum load has a significant effect upon the number of cycles to failure. This would be anticipated if the stresses are high enough to be within the range of significant rate of creep for the particular soil.

Figure 10 indicates the type of summary of information which has been obtained by Seed and co-workers in order to relate the sustained stress, the earthquake stresses, and the number of cycles. It is evident that this is quite similar to the steady stress vs. alternating stress diagram (Figure 2) which is used to describe fatigue failures of metals. However, for soils in the high stress range it is also necessary to introduce the number of cycles to failure. If 100 cycles to failure represents a satisfactory design criterion, then a relation as indicated on Figure 10

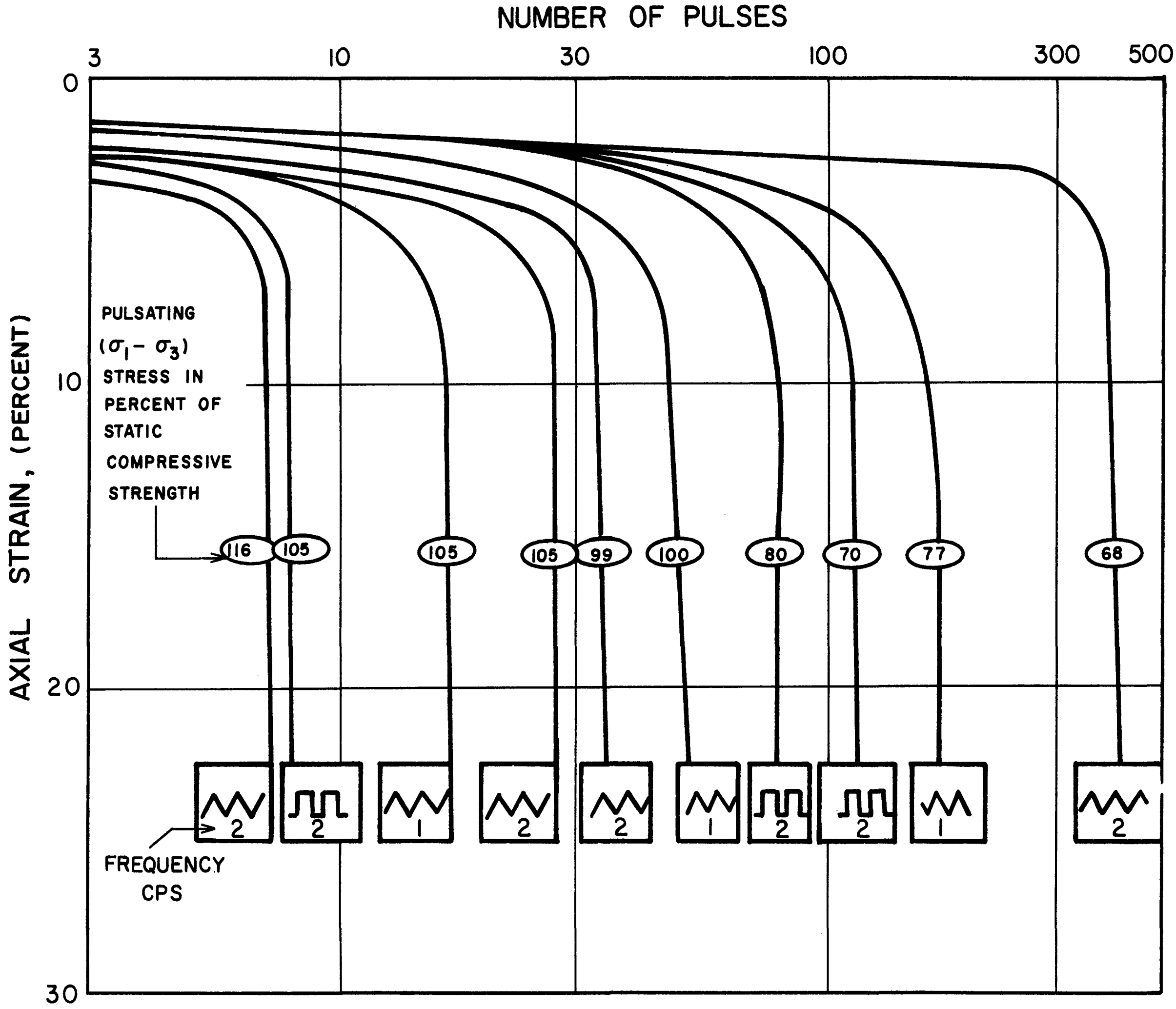


Figure 9. Results of Pulsating Load Tests on Samples of Silty Clay from L Street Slide Area, Anchorage (From Seed and Chan, 1964).

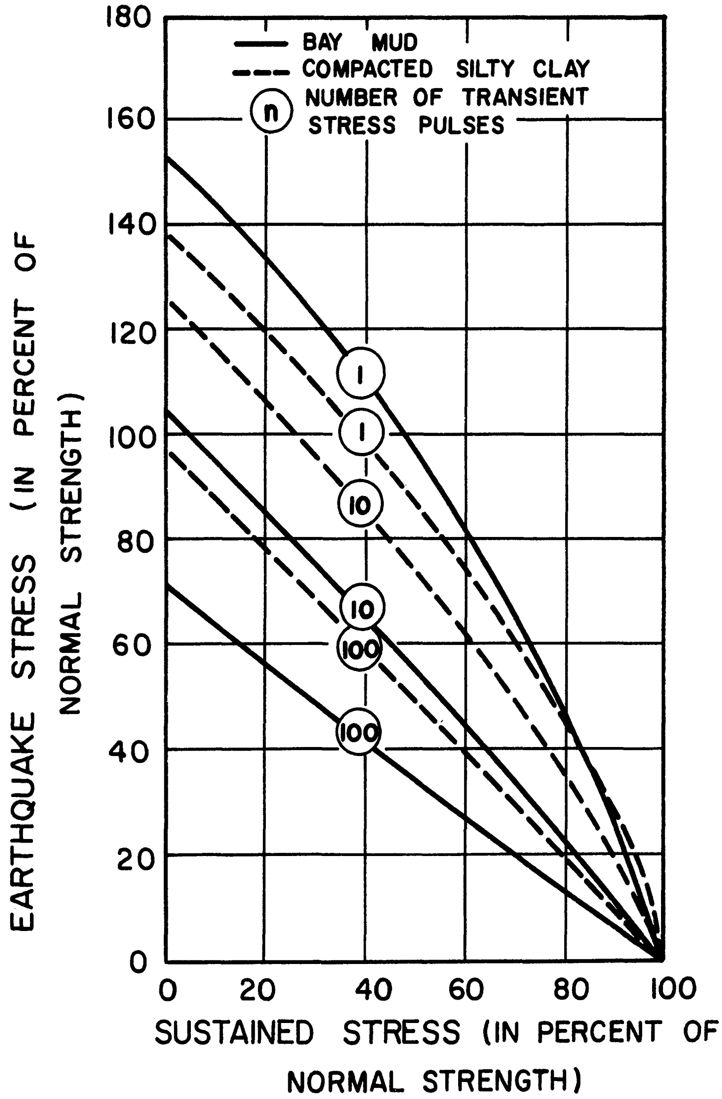


Figure 10. Comparison of Stress Conditions Causing Failure for Compacted and Undisturbed Samples of Silty Clay (From Seed, 1960).

could be established for a particular soil deposit. It is interesting to note that for the test results described by the solid lines on Figure 10, approximately 70 per cent of the normal strength could be applied 100 times before causing failure under the condition of zero sustained stress. The paper by Seed and Chan (1966) presents a comprehensive discussion of the strength of clays under earthquake loading conditions and, with the discussions which will surely follow in subsequent Journals of the Soil Mechanics and Foundations Division, ASCE, will constitute the most significant contribution to the study of cohesive soils under high amplitude-low cycle loading or earthquake-simulated conditions.

COMMENTS ON PAPERS PRESENTED TO THE SYMPOSIUM

Although the papers contributed to Session II, concerning the effect of repeated loading on soils and rocks, are available in their entirety and include summaries by their authors, it may be useful to include a few comments to indicate the writer's concepts of how these recent studies fit in with the preceding discussions. In the five papers presented, three include information regarding the dynamic behavior of cohesionless soils and three include information on similar behavior of cohesive soils.

The paper by Bustmante, Juarez and Cabrera describes an experimental study of the direct shear tests of sand which was compacted to minimum void ratio. The deformation of the shear box was strain-controlled and loading patterns of continuous loading, half cycle loading (loading up to a given magnitude of stress, then reducing the load to zero but with no reversal of deformation, then repeated application of the load in the same direction), and a third type of test in which the direction of shearing load was reversed after each application of maximum load. Under these restricted testing conditions they determined that the peak strength depended strongly on the maximum grain size but not upon gradation. They found little difference between the peak strength and stress-strain envelope between the continuous type of loading and the repeated half-cycle loading. They considered that the strength was not reduced even by a large number of half cycle loadings and that the strength depended upon the initial deformation. They concluded that the type of cycling should not affect the continuous strength even for large normal pressures. The test results obtained were interesting and warrant further study but the results are strictly applicable only to shear strength of dense sand tested in a confined situation under a strain-controlled type of loading.

The papers by Baker and Kraft, Miranda, and Calhoun deal with the dynamic behavior of soils associated with blast loading. Baker considered the effect of the stress-strain relations of soils on the shape of a transient stress pulse and, in particular, upon the rise time at the leading edge of this pulse and the reflection of this stress pulse from a rigid structure which intercepts the one-dimensional wave. One-dimensional wave propagation tests were performed on 20-30 Ottawa sand under conditions of a constant lateral stress, in order to produce a stress-strain curve of the type indicated by curve A, Figure 6, and in a confined condition designed to produce a strain-hardening type of stress-strain curve as indicated by curve C, Figure 6. A uniformly graded concrete sand was also tested in the confined condition. The tests indicate that for wave propagation in a strain-softening type of material a softening of the wave pulse front occurs. On curve A, Figure 6, the modulus of elasticity is indicated by the slope of the stress-strain curve and it is steeper for low strains than it is for high strains. Because the wave propagation velocity is a function of the effective modulus of elasticity, it might be anticipated that the low strains would propagate at a higher velocity thereby tending to flatten the pulse front or to increase the time of rise to maximum pulse stress. On the other hand for the strain-hardening material, the higher stress portion of the pulse propagates at a higher velocity thereby steepening the pulse front. The test results also show that the reflected stress level is less than twice the incident stress level for tests where a strain-softening stress-strain curve was involved and was more than twice the incident stress level in tests where the stress-strain curve was strain-hardening.

Kraft, Miranda, and Calhoun also considered the shock propagation phenomena in non-linear media. Much of their test information is described in terms of the ratio M_t/M_s or the ratio of the tangent modulus to the secant modulus. This ratio describes the characteristics of the stress-strain curve. When the tangent modulus is less than the secant modulus, the stress-strain curve is indicated as strain-softening, and when it is greater the stress-strain curve is strain-hardening. They have solved the differential equation relating peak stress intensity with time and have determined the peak stress attenuation with depth. Experimental tests were conducted using a clay-silt soil which was preloaded for approximately 20 minutes, just before the dynamic load was applied. Because there was no direct indication as to whether the consolidation was complete, or of the magnitude of the void ratio change during the time of pressure application, or of the time-pressure or time-displacement traces from the dynamic loading, is difficult to evaluate their non-dimensional stress parameters and the non-dimensional depth. It would be very useful to have the complete report for further study.

Kondner, Krizek and Haas have studied the dynamic stress-strain-time properties of a remolded plastic clay utilizing strain-controlled vibratory compression techniques. A sinusoidal input amplitude was applied and the dynamic force and phase angle between force and amplitude were measured. By gradually increasing the input amplitude and measuring the associated stress developed they were able to develop dynamic stress-strain curves which essentially indicated the restoring stress-strain behavior of the material. Different water contents of the clay produced different dynamic stress-strain curves but the authors were able to collapse these curves by introducing a dimensionless stress as the ordinate (dynamic stress divided by unconfined compression strength). They found little influence on the test results from changes in moisture content, static stress level, specimen geometry, frequency of oscillation, or strain rate amplitude within the limits of each utilized. However, as they note, this does not warrant extrapolation of these findings to other situations. For example, the frequency variation over the range from 5 to 100 cycles per second indicates that all of these frequencies were well above the values (1 per 2 minutes to 20 cycles per minute) used by Seed for which he did find a strain-rate effect. They did find a reduction in the total maximum stress to cause failure (the sum of the static stress and dynamic stress) at a value ranging from 60 to 75 per cent of the average unconfined compressive strength of the specimen. At failure the specimen would have been subjected to approximately 25,000 repetitions of the loading cycle and this indication of the order of magnitude of stress reduction by repeated loadings is consistent with the findings of previous investigators.

The paper by Caballero describes a method for estimating the behavior of flexible pavements for roads of first order in Mexico, designed for small truck traffic. The method includes repeated loadings up to 100 cycles of a rigid plate of 30.5 cm. diameter on the pavement surface and measurements of the permanent deformations. These plate test results are extrapolated to 10,000 or more repetitions and are also compared with observations of pavement behavior. The performance of a pavement is a function of many variables, for example, the pavement design, geological characteristics of the soil, weathering, load history, and water conditions. Thus, a plate loading test on the finished pavement produces empirical information on the behavior of the asphalt-base course-soil system. The other approach is to evaluate the behavior of the asphalt, base course, and soil, separately under repeated loadings and attempt to synthesize the results. Information concerning both methods of approach are included in the Highway Research Board Proceedings, publications of the various state and federal highway research laboratories, and proceedings of conferences similar to the International Conference on the Structural Design of Asphalt Pavements.

SUGGESTIONS FOR RESEARCH

The foregoing discussion indicates there are many topics related to the low-cycle, high stress amplitude behavior of soils which need further study. A few of these topics are mentioned below.

1. For a few cycles of high amplitude stress it is expected that compaction of the soil will occur. However, for conditions of low confining pressures and high input energy, loosening may occur. The boundary between compaction and loosening of the soil structure should be established for given soil conditions and energy levels.

2. Field evaluations are needed to correlate the theoretical estimates of the effect of geometry and laboratory values of soil behavior which relate to propagation of wave energy through soils.

3. The effects of a few cycles of high amplitude stress on subsequent response and propagation characteristics of soils under low amplitude conditions may be of importance.

4. Diagrams similar to Figure 10 should be prepared for different soils to include effects of stress repetitions up to 10^6 or 10^7 cycles. Possibly this information could be arranged to indicate cumulative strains as functions of the confining pressure, the alternating stress, the material characteristics, and stress history.

REFERENCES

Casagrande, A. (1936) "Characteristics of Cohesionless Soils and the Stability of Slopes and Earth Fills." J. Bost. Soc. C.E., Jan.

Casagrande, A. and Shannon, W. L. (1948) "Stress-Deformation and Strength Characteristics of Soils under Dynamic Loads" Proc. 2nd ICSMFE. Vol. V. pp 29-34.

Chen, L-S (1948) "An Investigation of Stress-Strain and Strength Characteristics of Cohesionless Soils by Triaxial Compression Tests" Proc. 2nd ICSMFE. Vol. V. pp 35-43.

Crandell, F. J. (1949) "Ground Vibration Due to Blasting and Its Effects on Structures" Jnl. Bost. Soc. C.E., April, 1949.

D'Appolonia, E. (1953) "Loose Sands Their Compaction by Vibro-Flotation" ASTM STP 156 Symposium on Dynamic Testing of Soils pp 138-154 disc 155-162.

D'Appolonia (1966) "Performance of Spread Footings on Vibratory Compacted and Preloaded Fine Sand" paper presented at ASCE Structural Engineering Conference, Miami, Florida January 31 - February 4.

Diaz de Cossio, R. (1960) "Foundation Failures During the Coatzacoalcos (Mexico) Earthquake of 26 Aug 1959" Proc. 2 WC Earthq. Eng., 1, p 473-486.

Falconer, B. H. (1964) "Niigata Earthquake, Japan, 1:02 p.m., 16 June, 1964" Internation Institute of Seismology and Earthquake Engineering, Tokyo, Japan, 41 pp.

Florin, V. A. and Ivanov, P. L. (1961) "Liquefaction of Sandy Soils" Proc. 5th ICSMFE, Vol. I. pp. 107-111.

Geuze, E.C.W.A. (1948) "Critical Density of Some Dutch Sands" Proc. 2nd ICSMFE. Vol. III pp 125-130.

Jackson, J. G., Jr. and Hadala, P. F. (1964) "Dynamic Bearing Capacity of Soils. Report 3: The Application of Similitude to Small-Scale Footing Tests" U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss. Tech. Rep. No. 3-599 Rep. 3. Dec.

Kummeneje, O. and Eide, O. (1961) "Investigation of Loose Sand Deposits by Blasting", Proc. 5th ICSMFE Vol. I pp. 491-497.

Lyman, A. K. B. (1942) "Compaction of Cohesionless Foundation Soils by Explosives", Trans. ASCE, Vol. 107. pp. 1330-1348.

Marsal, R. J. (1961) "Behavior of a Sandy Uniform Soil During the Jaltipan Earthquake, Mexico", Proc. 5th ICSMFE, Vol. I, pp. 229-233.

Murayama, S. and Hata, S. (1957) "On the Effect of Remoulding Clay", Proc. 4th ICSMFE, Vol. I. pp. 80-82.

Nash, K. L. and Dixon, R. K. (1961) "The Measurement of Pore Pressure in Sand Under Rapid Triaxial Test", Proc. of the Conf. on "Pore Pressure and Suction in Soils", 1960, Butterworths, London.

Newmark, N. M., Mosborg, R. J., Munse, W. H., and Elling, R. E. (1951) "Fatigue Tests in Axial Compression", Proc. ASTM, v. 51, p. 792.

Prugh, B. J. (1963) "Densification of Soils by Explosive Vibration", J. Constr. Div. Proc. ASCE, v. 89, No. CO1. pp. 79-100.

Seed, H. B. (1957) Discussion, Proc. 4th ICSMFE, Vol. III. pp. 86-87.

Seed, H. B. (1960) "Soil Strength During Earthquake", Proc. 2nd World Conf. Earthquake Eng., Vol. 1, pp. 183-194.

Seed, H. B. and Lundgren, R. (1954) "Investigation of the Effect of Transient Loading on the Strength and Deformation Characteristics of Saturated Sands", Proc. ASTM, Vol. 54, pp. 1288-1306.

Seed, H. B. and Chan, C. K. (1964) "Pulsating Load Tests on Clays" Appendix D of Shannon and Wilson Report on "Anchorage Area Soil Studies, Alaska".

Seed, H. B. and Chan, C. K. (1966) "Clay Strength under Earthquake Loading Conditions", J. Soil Mech and Found. Div., Proc. ASCE, Vol. 92, No. SM2, March. pp. 53-78.

Seed, H. B. and Lee, K. L. (1964) "Pulsating Load Tests on Sands", Appendix C of Shannon and Wilson Report on "Anchorage Area Soil Studies, Alaska".

Shannon and Wilson, Inc. (1964) "Anchorage Area Soil Studies Alaska". Report to U.S. Army Engineer District, Anchorage, Alaska.

Soderberg, C. R. (1933) "Working Stresses," Trans ASME, Appl. Mech. Div., Vol. 1, No. 3, APM 55-16.

Steinbrugge, K. V. and Clough, R. W. (1960) "Chilean Earthquake of May, 1960, A Brief Report", Proc. 2nd World Conf. Earthquake Eng., Vol. 1 pp. 629-637.

Steuerman, S. (1939) "A New Soil Compaction Device" Eng. News Record, July 20.

Taylor, D. W. and Whitman, R. V. (1954) "The Behavior of Soils Under Dynamic Loadings" - 3. Final Report on Laboratory Studies, Aug. MIT Dept. of Civil and Sanitary Eng. Soil Mechanics Lab.

Wöhler, A. (1858) "Uber die Festigkeitsversuche mit Eisen und Stahl", Z. für Bauwesen, v. 8.

Yao, J. T. P. and Munse, W. H. (1962) "Low-Cycle Fatigue of Metals - Literature Review", The Welding Journal 41:4, Research Supplement, pp. 182s - 192s .

Yao, J. T. P. and Munse, W. H. (1963) "Low-Cycle Fatigue Behavior of Axially Loaded Specimens of Mild Steel", Ship Structure Committee SSC-151, Distr. by U.S. Dept. Commerce, Office of Technical Services, Wash. 25, D. C.

Papers for International Symposium on the Effects of Repeated Loading of Materials and Structures - Session II - Effects on Soils and Rocks. RILEM Mexico City, September 15-17, 1966.

Bustamante, J. I., Juarez, E., and Cabrera, A. (1966) "Shear Strength of Sand Subjected to Slow Loading Cycles".

Baker, W. J. (1966) "Characteristics of Stress-Wave Interaction with Underground Structures".

Kraft, D. C., Miranda, C.X.C.F., and Calhoun, D. E. (1966) "Shock Propagation Phenomena in Nonlinear Media".

Kondner, R. L., Krizek, R. J., and Haas, H. J. (1966) "Dynamic Clay Properties by Vibratory Compression".

Corro Caballero, S. (1966) "Estimation of Flexible Pavement Performance through Rigid Plate Repetitive Static Load Tests".