Soil Dynamics and Its Application to Foundation Engineering

Dynamique du Sol et son Application aux Travaux de Fondations

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FOREWORD

This is the first time that soil dynamics has been accorded full status as a main session at the ICSMFE, although it was covered in specialty sessions at the Seventh Conference in Mexico City (organized by Prof. R.V. Whitman) and at the Eighth Conference in Moscow (organized by Prof. S. Prakash).

The remarkable progress in soil dynamics during the past decade may be attributed to a timely combination of the need and our capabilities to meet it. The need has arisen from the increase in construction activities in general. For example, the advent of nuclear power plants has created a need to look into the soil-structure interaction problems during earthquakes more carefully even in areas of low seismicities. The costly lessons learned from the Alaskan and Niigata earthquakes of 1964 have obviously given a powerful impetus to earthquake oriented research. There have also been other problems such as offshore oil storage tanks subjected to ocean wave loads, environmental problems involving ground vibrations, etc.

Our recently acquired capabilities to solve soil dynamics problems in analyses and experiments owe their effectiveness to the remarkable progress in digital computers and electronic measuring and recording systems. The analytical capabilities have stimulated experimental research to seek more accurate stress-strain relationships of soils and rocks.

Y. Yoshimi, General Reporter, is responsible for the organization and editing of this report, and for the writing of Chapter 2. F.E. Richart, Jr., wrote Chapters 1 and 5, and assisted Yoshimi in the editing. Chapter 3 was written by S. Prakash, and Chapter 4 by D.D. Barkan and V.A. Ilyichev. Although originally planned, the topic of vibratory compaction could not be completed in time for this report, but is expected to be included in the General Report in Proceedings Volume 3. Emphasis has been placed on the literature published since the Mexico Conference of 1969. Even so, the progress has been so rapid that some significant papers might admittedly be missing from the list of references.

1. DYNAMIC STRESS-STRAIN RELATIONSHIPS FOR SOILS (by F.E. Richart, Jr.)

1.1 Introduction

Dynamic loadings develop forces in soils which may modify the conventional static stress-strain relationships. These modified relationships are required for dynamic response analyses of soil masses or for dynamic soil-structure interaction studies in which time-dependent motions are considered. This chapter treats recent developments covering dynamic deformations of soils and the factors which influence them: it includes principally material developed since the Specialty Conference on Soil Dynamics (1969). Additional summaries or state of the art papers covering publications up to about 1970 were given by Alpan (1970), Krizek (1971), Richart et al (1970), and Whitman (1970), for example. Failure conditions by liquefaction are considered in Chapter 2.

The shape of the stress-strain relationship for any particular soil depends upon the type of loading and boundary restraining conditions. Figure 1.1 shows typical behaviors of cylindrical samples of cohesionless soil when subjected to an increasing axial stress, σ_z, while a radial pressure, σ_r acts on its periphery. Curve A represents hydrostatic compression ($\sigma_r = \sigma_z$), a loading condition which develops a "strain-hardening" nearly elastic nonlinear type of stress-strain curve. Curve B represents the condition of constrained compression (no radial expansion permitted at the periphery of the sample) which is again a strain-hardening type of curve, but tests show an appreciable hysteresis loop developed upon unloading. Curve C illustrates the strain-softening type of stress-strain curve developed for a constant restraining boundary pressure as developed in a stress-controlled triaxial test.

At very low values of strain, the initial portion of curves A, B, or C may be approximated by a *linear* elastic stress-strain curve. One of the objectives of this discussion is to describe methods of evaluating the linear "elastic moduli" for soils and to note how these moduli are modified for dynamic loadings. Another objective is to present results of laboratory and field tests concerning dynamic stress-strain curves of type C.

1.2 Equations for Stress-Strain Relationships

Because the different shapes of the stress-strain curves in Fig. 1.1 are produced by different proportions of volumetric compression and shearing strains, it is sometimes convenient to express the general stress-strain relations in the form of a constitutive equation. For example, Jackson (1969) has shown how laboratory test data can be interpreted to fit into an equation of the

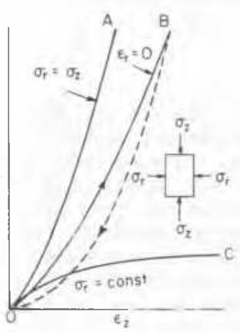

Fig.1.1 Stress-Strain Curves for Triaxial Tests of Sand with Various Lateral Restraints

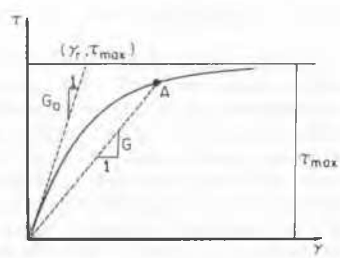

Fig.1.2 Basic Parameters for Hyperbolic Stress-Strain Curves

type,

$$\sigma_{ij} = K \bar{e} \delta_{ij} + 2G(\epsilon_{ij} - \frac{1}{3} \bar{e} \delta_{ij}) \qquad (1.1)$$

and a yield condition

$$\sqrt{J_2''} = f(\bar{\sigma}_o) \qquad (1.2)$$

to describe dynamic behavior of soils under ground shock loadings. In Eq.(1.1), σ_{ij} = total stress tensor; ϵ_{ij} = total strain tensor; K = modulus of volume compressibility, or bulk modulus; $\bar{e} = \epsilon_x+\epsilon_y+\epsilon_z$ = cubic dilation, or volumetric strain; G = shear modulus; and δ_{ij} = Kronecker delta function (δ_{ij} =1 when i=j, δ_{ij}=0 when i≠j). In Eq.(1.2), J_2'' = second invariant of the stress deviation, and $\bar{\sigma}_o$ = average normal effective stress or octahedral normal stress.

Constitutive equations similar to Eq.(1.1) have often been incorporated into analytical studies of blast loadings on soils in which the soil was considered to deform primarily in one-dimensional compression (constrained compression). Both volumetric compression and change of shape are developed in constrained compression and the general stress-strain curve is similar to curve B of Fig. 1.1. However, for "low stress" levels (below about 3.5 kg/cm^2) Hadala (1973) noted that nonlinear curves of the strain-softening type were developed for many soils. This effect depended upon the type of soil, its initial relative density or degree of compaction, and the degree of saturation. Thus, the strain-softening type of stress-strain curve may be developed in most practical soil dynamics problems.

Theoretically, elastic shearing stress-strain relations developed by pure shear involve no change of volume, which causes $\bar{e}$=0 in Eq. (1.1). This leaves only the linear relation between shearing stress and shearing strain, which may be represented by

$$\tau_{xz} = G \gamma_{xz} \qquad (1.3)$$

in the x-z plane (after converting the tensor symbols to ISSMFE symbols). Equation (1.3) relates shearing stress to shearing strain through the shear modulus, G. This expression may also be used to represent conditions at any point along a shearing stress-shearing strain curve, as shown in Fig. 1.2 for the strain-softening type curve. At shearing strains approaching zero, the value of G is a *maximum* and is designated as G_o. At a larger value of shearing strain, for example at a strain corresponding to point A in Fig. 1.2, the line connecting the origin to A represents the secant modulus, designated as G. The value of the secant modulus decreases as the shearing strain increases.

The shape of the strain-softening shearing stress-strain curve for a particular soil can be adequately represented by a hyperbolic curve (Kondner, 1963, Hardin and Drnevich, 1972a, 1972b) which has a slope of G_o at γ= 0 and which is asymptotic to the horizontal line representing $\tau = \tau_{max}$ (Fig. 1.2). This curve is defined by the equation

$$\tau = \frac{\gamma}{1/G_o + \gamma/\tau_{max}} = \frac{\gamma G_o}{1 + \gamma/\gamma_r} \qquad (1.4)$$

in which γ_r = "reference shearing strain"= τ_{max}/G_o. Hardin and Drnevich found that, by presenting test data in the normalized form of τ/τ_{max} vs γ/γ_r, data from clay specimens fell nearly on a single curve while data from sand specimens fell along another curve. Neither curve fit the hyperbolic shape exactly, but by devising a "hyperbolic strain" parameter, γ_h, they were able to adjust the curves for sand and for clay to fit on one curve on a τ/τ_{max} vs γ_h plot.

The same hyperbolic type curves have been obtained from static and dynamic torsion tests. Test results from reversed torsion loading of a sand sample are shown in Fig. 1.3. Two complete hysteresis loops are shown, one with extremities at A-A' and a "hysteresis modulus" of G_1, and the second with extremities at B-B' and a hysteresis modulus G_2 which is smaller than G_1 because of the larger shearing strain reached at point B. The dots along the initial loading curve

(OAB) represent extremities of hysteresis loops for other maximum strain values. Test data consistently show hysteresis loops which increase in width (increased loss of energy) and exhibit lower secant or hysteretic moduli as the maximum shearing stress in the loading cycle increases.

Equation (1.4) is adequate to describe the loading curve (OAB) in Fig. 1.3, but it is difficult to represent the hysteresis loops by hyperbolic segments. A convenient mathematical relation to describe the loading curve OAB is given by

$$\frac{\gamma}{\gamma_r} = \frac{\tau}{\tau_{max}}\left[1 + \alpha\left|\frac{\tau}{C_1\tau_{max}}\right|^{R-1}\right] \qquad (1.5)$$

(Ramberg and Osgood, 1943) in which α, R, and C_1, are constants which permit adjustments of the shape and position of the curve. For

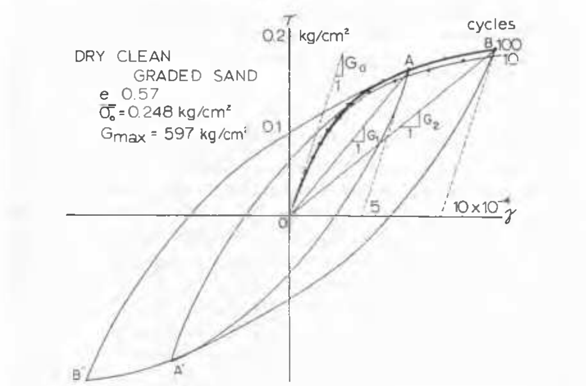

Fig.1.3 Stress-Strain Hysteresis Loops for Reversed Loading (Hardin and Drnevich, 1972b)

unloading from conditions (τ_1, γ_1) the Ramberg-Osgood (R-O) curve follows the expression

$$\frac{\gamma-\gamma_1}{\gamma_r} = \frac{\tau-\tau_1}{\tau_{max}}\left[1 + \alpha\left|\frac{\tau-\tau_1}{2C_1\tau_{max}}\right|^{R-1}\right] \qquad (1.6)$$

Ramberg-Osgood curves have been used for more than a decade to represent the stress strain behavior of structural elements, and more recently (Constantopolous, et al, 1973; Streeter, et al, 1974; Richart and Wylie, 1975; and Faccioli and Ramirez, 1976) the R-O curves have been adopted to represent dynamic stress-strain behavior of soils.

1.3 Shearing Strength under Dynamic Loading Conditions

Figure 1.2 and Eq.(1.4) illustrate that the two soil properties needed to evaluate the dynamic shearing stress-strain curve for a particular soil are the small strain shear modulus, G_o, and the ultimate shearing strength, τ_{max}. Both of these quantities must be evaluated for loading and restraining conditions anticipated for the soil in situ.

It is often convenient to establish the maximum shearing stress for a given soil under static loading conditions, then add corrections to account for dynamic effects. Under static conditions the maximum value of shearing strength available on a given plane through a soil mass depends (according to the Mohr-Coulomb failure theory) upon the normal pressure acting on that plane, the effective angle of internal friction, ϕ', and the cohesion intercept, c'. Of course, the shearing strength should be evaluated on the plane which will be subjected to the maximum static plus dynamic shearing stresses. For example, for vertical shear stress propagation through layers of soil, as is often considered in earthquake excitations, the maximum allowable shearing stress on the horizontal (and vertical) planes in a soil mass is given by

$$\tau_{max} = \left\{\left[\left(\frac{1+K_o}{2}\right)\bar{\sigma}_v \sin\phi' + c'\cos\phi'\right]^2 - \left[\frac{1-K_o}{2}\bar{\sigma}_v\right]^2\right\}^{1/2} \qquad (1.7)$$

in which $\bar{\sigma}_v$ is the effective vertical stress, and K_o is the coefficient of earth pressure at rest $(\bar{\sigma}_h = K_o\bar{\sigma}_v)$. The vertical stress on any plane is the total effective stress produced by the weight of the overburden plus pressures developed by construction of a building, etc.

1.3.1 Strain Rate Effects

Tests to determine the increase in τ_{max} caused by an increase in rate of loading above normal static loading rates have been conducted during the past three decades by utilizing dynamic loading to failure in unconfined, triaxial, direct, or simple shear tests. The test results are described herein as $(\tau_{max})_{dynamic} = (\tau_{max})_{static}$ x (strain rate factor). These results demonstrating the "strain rate effect" have been summarized by Whitman (1970) and Richart, et al (1970) who noted that: (a) for dry sands the strain rate factor was less than 1.10 to 1.15 for strain rates varying from about 0.02% per second to 1000% per second; (b) for saturated cohesive soils the strain rate factor was 1.5 to 3.0; and (c) for partially saturated soils the strain rate factor was 1.5 to 2.0. Dynamic triaxial tests on dry sand by Lee, et al (1969) showed a strain rate factor, for strain rates varying from 0.1% to 10000% per minute, of: (a) about 1.07 for loose sand at confining pressures up to 15 kg/cm^2 and for dense sand at low confining pressure (less than about 6 kg/cm^2), and (b) about 1.2 for dense sand at high confining pressures.

From this brief discussion it is evident that the strain rate effect is relatively unimportant for dry sands, but may be significant for clays at high rates of strain. For saturated sands and silty material the effect of strain rate includes the time-dependent build up of pore pressures and possible liquefaction.

1.3.2 Effect of Repeated Loadings

The strain rate effect described in the preceding section indicated that a maximum stress greater than the static maximum stress can be developed in the sample if an increasing load is applied rapidly. However, if stresses less than this dynamic maximum are repeated enough times, failure of the sample will occur. This constitutes a low-cycle fatigue type of failure which has been described by Seed and Chan (1964) and Murayama and co-workers (see Murayama 1970, for example) for clays. The number of repetitions of a particular stress level which may be applied before failure depends on the initial sustained stress level, the shape of the repeated stress pulse, the frequency of pulse application, the type of testing device, and of course, on the characteristics of the soil being tested.

Similar low-cycle fatigue behavior has been observed in repeated triaxial tests of saturated sands, but here the failure was usually developed by liquefaction. A special note should be made with respect to the reduction of the shearing strength of saturated sands by an increase in pore pressure, whether it is static or dynamic. An increase in pore pressure reduces the value of $\bar{\sigma}_o$ in Eq.(1.2) or $\bar{\sigma}_v$ in Eq.(1.7) and a lower value shearing strength is developed. Thus a given applied shearing stress will represent a greater proportion of the available shearing strength as pore pressures increase.

Repeated loadings of dry sand may lead to an actual strengthening of the material, because of reductions of void ratio and more favorable grain orientations, but greater deformations may be developed than for the static case because of these progressive movements (see Toki and Kitago, 1974).

The shearing strengths of sands and clays are reduced if they are tested under conditions of increasing shear stresses plus superposed vibratory shearing stresses, or increasing shearing stresses while the normal stresses on the failure plane are subjected to vibratory pulsations. Tests of this type have been reported by Barkan (1962), Ermolaev and Senin (1968), and Satyavanija and Nelson (1971). The reduction in shearing stress is a function of the static confining pressures, magnitude of the vibratory stress, frequency and duration of the vibratory stresses, and the type of soil.

This brief discussion of the effects of strain rate and repeated loads on shearing strength of soils is presented to emphasize again that the numerical value of shearing strength to be included in analytical studies of dynamic soil motions must be determined for conditions likely to be encountered in the field.

1.4 Low-Amplitude Shear Modulus, G_o

Because soils exhibit a nonlinear stress-strain behavior, it is necessary to establish what value of shearing strain represents "low amplitude" or at what limiting value is there negligible change in G_o. Laboratory test results have shown that there is insignificant change in v_s for tests run at shearing strain amplitudes of 10^{-5} or less. Seismic tests usually develop shearing strains in the field of 10^{-6} or less, therefore these tests develop low amplitude strains. Laboratory tests of the resonant column type may also be controlled to develop shearing strains on the order of 10^{-6}, which permits an opportunity to check field values against values of G_o determined in the laboratory.

1.4.1 Evaluation of G_o by Field Tests

Seismic measurements can establish values of the shear wave velocity, v_s, within a rock or soil mass. Then the low amplitude shear modulus is calculated from

$$G_o = \rho\, v_s^{\,2} \tag{1.8}$$

in which ρ is the mass density (unit weight/g) of the soil. General seismic investigation procedures are described in textbooks and this information may be supplemented by technical publications directed toward shear wave evaluations (for example, Mooney, 1974; and Ballard and McLean, (1975), for soils in general, Roethlisberger (1972) for frozen soils, and Hamilton (1971) for submarine soils). Large zones of subsurface soil or rock can be explored from the surface using shear wave refraction or steady state (Rayleigh wave) techniques. For detailed information on the variation of wave velocities with depth at a particular location, bore hole techniques can be used. Up-hole and down-hole tests can be performed with one bore hole while cross-hole tests require two or more bore holes. In the up-hole method, the excitation is provided at various depths within the bore-hole and the sensor is placed at the surface while for the down-hole method the excitation is applied at the surface and one or more sensors are placed at different depths within the hole (see Figs. 1.4a and 1.4b for arrangement of equipment). Both the up-hole and down-hole method give average values of wave velocities for the soil between the excitation and the sensor if one sensor is used, or between sensors if more than one are located in the bore hole.

In the cross-hole method (Stokoe and Woods, 1972), at least two bore holes are required, one for the impulse and one or more for sensors. As shown in Fig. 1.4c the impulse rod is struck at the top end and an impulse travels down the rod and is transmitted to the soil at the bottom. This shear impulse creates shear waves which travel horizontally through the soil to the vertical motion sensor located in the second hole, and the time required for the shear wave to traverse this known distance is measured. Because the distance between the impulse and pickup sensor is a critical quantity in evaluating v_s, it is necessary to determine the deviation from verticality of the bore holes by using a Slope Indicator or similar device.

The cross-hole method is relatively cheap and easy to use, and by reversing the direction of impulse on successive tests (Schwarz and Musser, 1972) the shear wave signal may be enhanced for more precise identification of the shear wave. It is possible to run cross-hole tests in holes previously drilled and cased if a large shear impulse is developed by a drop hammer acting against an anvil jacked tightly against the casing, but packing between the casing and soil may introduce an error in the measurement of v_s (Stokoe and Abdel-razzak, 1975). Recent developments in the cross hole technique (Miller, et al, 1975) now permit evaluation of v_s at large strain amplitudes.

1.4.2 Evaluation of G_o by Laboratory Tests

Dynamic tests using pulse techniques (see Whitman, 1970, for a description of tests) or resonant column equipment (see Richart et al, 1970, or Skoglund, et al, 1975 for description of test equipment) are most often used for laboratory determinations of G_o.

The resonant column method has been available for determining wave propagation velocities in soil samples for four decades, but major developments have occurred during the past 18 years. More than 50 laboratories in the U.S.A., Canada, Japan, Mexico, Germany, Venezuela, and Turkey now use resonant column devices for research or soil investigation. In the resonant column test, a cylindrical(solid or hollow) column of soil is contained within a rubber membrane, placed in a triaxial cell, and set into motion in either the longitudinal or torsional mode of vibration. The frequency of the electro-magnetic drive system is changed until the first mode resonant condition is determined. This resonant frequency, the geometry of the sample, and the conditions of end restraint provide the necessary information to calculate the velocity of wave propagation in the soil under the given testing conditions. During the past decade it has often been found convenient to use hollow cylindrical samples, particularly for cohesive soils, in tests including shearing strain amplitude as one of the test variables.

In research investigations, it is always desirable to change only one test variable during each series of tests. Hardin and Black (1968) have noted the quantities which exert an influence on the shear modulus of soils and have expressed these as a functional relationship,

$$G = f(\bar{\sigma}_o, e, A, t, H, f, C, \theta, \tau_o, S, T) \qquad (1.9)$$

in which $\bar{\sigma}_o$ = average effective confining pressure; e = void ratio; A = amplitude of shearing strain; t = secondary effects that are functions of time and magnitude of time and magnitude of stress increment; H = ambient stress history and vibration history; f = frequency of vibration; C = grain characteristics; θ = soil structure, τ_o = octahedral shearing stress; S = degree of saturation, and T = temperature. Of course,

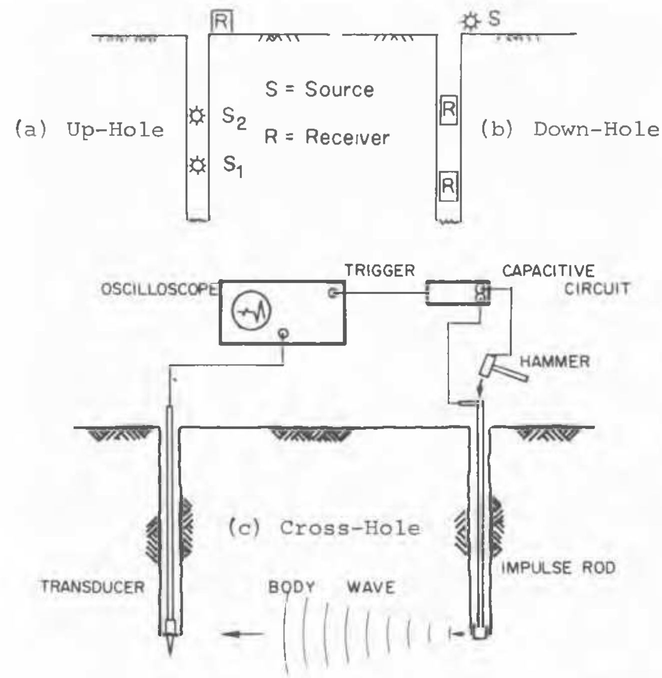

Fig.1.4 Field Test Procedures to Evaluate v_s

several of these quantities may be related (for example e, C, and θ). When discussing the *low amplitude* shear modulus, G_o, the effect of A is eliminated and we have only the remaining parameters to consider.

For clean sands, it has been found that G_o is essentially independent of each of the variables except $\bar{\sigma}_o$ and e. This conclusion has been confirmed by Kuribayashi, et al (1974). Also the conclusion that particle size, shape, and distribution has negligible effect on the wave propagation velocity in sands was confirmed by Krizek, et al (1974). Analytical expressions have been presented for the shear modulus of clean sands as

$$G_o = 700 \frac{(2.17-e)^2}{1+e} (\bar{\sigma}_o)^{0.5} \qquad (1.10)$$

for round-grained sands ($e<0.80$) and as

$$G_o = 326 \frac{(2.97-e)^2}{1+e} (\bar{\sigma}_o)^{0.5} \qquad (1.11)$$

for angular grained sands. In Eqs.(1.10 and 1.11), G_o and $\bar{\sigma}_o$ have units of kg/cm^2. Both equations were originally established to correspond to shearing strains of 10^{-4} or less. Equation (1.10) was found to give values slightly lower than those obtained by pulse tests (Whitman and Lawrence, 1963), and recently Iwasaki and Tatsuoka (1977) have determined experimentally that

$$G_o = 900 \frac{(2.17-e)^2}{1+e} (\bar{\sigma}_o)^{0.38} \qquad (1.12)$$

from tests on clean sands ($0.61<e<0.86$ and $0.2<\bar{\sigma}_o<5$ kg/cm^2) at shearing strain amplitudes of 10^{-6}. For shearing strains of 10^{-4} their results agreed with Eq.(1.10).

Equation(1.11) was modified slightly and proposed by Hardin and Black (1968) for use as a first estimate for the value of G_o for cohesive soils, corresponding to 1-day's duration of the confining test pressure. However, the *time effect* (t) has been found to be important for resonant column tests of fine-grained soils (Afifi and Woods, 1971; Marcuson and Wahls, 1972; Afifi and Richart, 1973; and Anderson and Woods, 1976). Figure 1.5 illustrates a typical increase in v_s with time at constant confining pressure, $\bar{\sigma}_o$. Note that v_s was evaluated intermittently during the test by vibrating the sample less than 30 seconds to obtain each reading. The test curve in Fig. 1.5 shows two distinct zones, a primary behavior corresponding to that which might be anticipated during reconsolidation of the sample to its in-situ stress condition and the secondary behavior or "secondary time effect" which is perhaps analogous to secondary compression. It is important to note that it requires from 100 to 1000 minutes for the primary reconsolidation to be complete in cylindrical cohesive soil samples 3.57 cm in diameter. Larger samples would require greater times for the primary behavior to be complete.

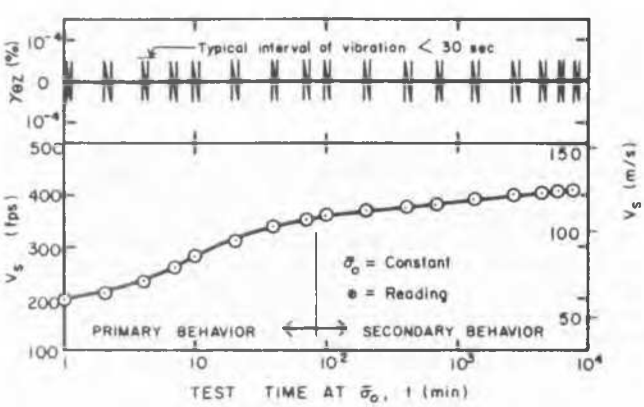

Fig.1.5 Time Effect in Resonant Column Tests (Anderson and Woods, 1976)

The secondary time effect has been described in terms of the increase in v_s (i.e., Δv_s) per log cycle of time expressed as a ratio of the value of v_s found at a cumulative testing time of 1000 minutes. Thus the ratio $\Delta v_s/v_{s1000}$ expresses the influence of the secondary time effect for different soils. When comparing laboratory test data to field conditions it is necessary to add to v_{s1000} the ratio $\Delta v_s/v_{s1000}$ multiplied by the number of log cycles corresponding to the time since the last major stress change occurred in the field conditions. For example, if a fill had been placed 20 years ago on a layer of soil where field v_s values were recently obtained, better agreement between laboratory and field values of v_s are produced if v_{s1000} is multiplied by $(1 + 4\ \Delta v_s/v_{s1000})$. The secondary time effect was found to be unimportant for soils having $D_{50}>0.04$ mm., but $\Delta v_s/v_{s1000}$ was found to be as large as 17% for some clays. In every laboratory test for v_s in cohesive soils, the secondary time effect must be evaluated (see Anderson and Woods, 1976 for data on secondary time effects).

The effect of temperature (T) of clay during resonant column testing was found to be unimportant by Anderson and Richart (1974). Tests of seven cohesive soils (total testing time of about 1 year) at 4°C and 22°C showed that v_s at 4°C was equal to or not more than 12% greater than v_s determined at 22°C. However, resonant column tests of frozen soils (Stevens, 1975) have shown a significant effect of changes in temperature near the freezing point.

1.4.3 Comparison of G_o (or v_s) from Field and Laboratory Tests

Pulse tests results from laboratory samples were compared with seismic field test results by Dobry and Poblete (1969) for a deposit primarily of basaltic sands and silts and a good agreement was found. Seismic and resonant column test results were obtained from clay soils in the Texcoco Basin (Martinez, et al, 1974) and higher values were found by the seismic method, with the difference increasing with depth of the sample. Resonant column and seismic results from limestone (Yang and Hatheway, 1976) showed appreciable differences. Possible sources of error were attributed to refraction through adjacent harder layers in seismic tests, possible nonhomogeneities in the laboratory samples, and time effects. Cunny and Fry (1973) reported on laboratory and field evaluations of G_o at 14 sites which included a variety of soils. The field method for evaluating G_o was the steady state surface vibration (Rayleigh wave method) and the resonant column test was used in the laboratory. From evaluation of their test data they found that the laboratory-determined shear and compression moduli ranged within ±50% of the in situ moduli. Discussions of this paper pointed out that the cross-hole method should give better values of v_s at the depth the undisturbed samples were taken, and that including the secondary time effect would bring the laboratory values for cohesive soils nearer to the field values. For tests of sands the secondary time effect is negligible, and Stokoe and Richart (1973) and Iwasaki and Tatsuoka (1977) have found good agreement between resonant column and cross-hole field test values. For cohesive soils, Trudeau et al (1973) and Anderson and Woods (1975) found agreement between resonant column and cross-hole test values of v_s after the secondary time effect correction was added to the laboratory values. Figure 1.6 shows the comparison of laboratory and field values of v_s, with the open symbols representing the 1000 minute value for laboratory data and the solid symbols showing the secondary time correction to the field history of 20 years.

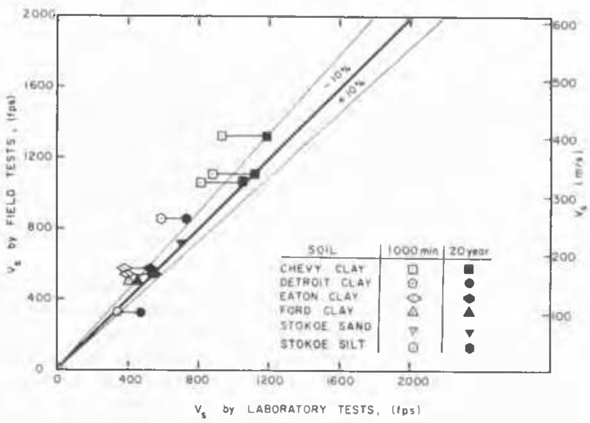

Fig.1.6 Comparison of Field and Laboratory Values of v_s (Anderson and Woods, 1975)

Thus, laboratory test values of v_s from the resonant column test involving shearing strains of 10^{-6} to 10^{-5} in undisturbed samples should be expected to agree within about 10% of the values of v_s obtained at the same location by cross-hole tests, if the secondary time correction is evaluated and applied to the laboratory test data.

1.5 Shape of the Shearing Stress-Strain Curve

Hardin and Drnevich (1972a) found that curves similar to that shown in Fig. 1.2 could adequately represent the torsional shearing stress-strain relations for sands and clays. If this test information was presented in dimensionless form with τ/τ_{max} as ordinate, and γ/γ_r as abscissa then all data for sands fell on one curve and the data for clays fell on a second curve. The slope of the secant to any point on one of these curves represents G/G_o, which reduces as the shearing strain increases. For evaluating the reduction in shear modulus because of earthquake induced strains in soils,it is convenient to plot G/G_o vs. γ/γ_r. Figure 1.7 includes two solid curves which describe the reduction in shear modulus as the shearing strain increases, for sands and for clays. Also shown in Fig. 1.7 is a dashed curve representing the average of data presented by Seed and Idriss (1970) for cohesive soils.

Often the numerical value of the shearing strain is used as abscissa and G/G_o as ordinate when presenting experimental results. This may lead to errors in interpretation, particularly for cohesionless soil. Consider a clean dry quartz sand for which e = 0.65, ϕ = 30°, and K_o = 0.6. A shearing strain of γ = 0.001 is developed in the sand mass, and we need to estimate the G/G_o values at a depth of 5m and 25m. From Eqs.(1.7) and (1.10) values of τ_{max} and G_o are calculated, then γ_r, γ/γ_r and G/G_o are as shown below:

Depth (m)	τ_{max} (kg/cm^2)	G_o (kg/cm^2)	γ_r	γ/γ_r	G/G_o
5	0.28	750	3.7×10^{-4}	2.70	0.37
25	1.40	1690	8.2×10^{-4}	1.22	0.61

Because the ratio of τ_{max}/G_o increases as a function of $(\bar{\sigma}_o)^{0.5}$ the reference shearing strain increases with depth and the chosen strain of γ =0.001 produces a smaller γ/γ_r at the deeper location. Therefore the value of G/G_o is larger at the greater depths. The influence of confining pressure on G/G_o was also noted by Shibata and Soelarno (1975) who developed an equation for the secant modulus reduction of sands at increasing shearing strains as

$$\frac{G}{G_o} = \frac{1}{1 + 1000 \dfrac{\gamma}{(\bar{\sigma}_o)^{0.5}}} \tag{1.13}$$

Note that Eq.(1.13) is quite similar to Eq.(1.4) since $\gamma_r = f(\bar{\sigma}_o)^{0.5}$. Thus Eq.(1.13) represents a modified hyperbola, comparable to the one developed by Hardin and Drnevich (1972b). The Shibata equation was found to fit recent test data (Yoshimi, 1976), and interpretation of this information led to development of the dotted curve shown in Fig. 1.7 which corresponds closely to the Hardin-Drnevich curve for sands.

In addition to resonant column tests, dynamic simple shear tests of sands (Silver and Seed, 1971, Park and Silver, 1975) free vibration and forced simple shear tests on clay (Kovacs et al, 1971), free torsional vibration tests of cohesive soils (Taylor and Parton, 1973; Zeevaert, 1973), ring torsion dynamic tests (Yoshimi and Oh-Oka, 1973), and high amplitude cross-hole field tests (Miller, et al, 1975) have given results showing the reduction of shear modulus with increasing shearing strain. From resonant column test results, Anderson and Richart (1976) found G/G_o vs γ/γ_r relations for six cohesive soils, which could be adequately represented throughout the range of $10^{-5}<\gamma<10^{-3}$ by a Ramberg-Osgood curve with parameters α=1,C,=0.4, and R=3. This R-O relation is shown in Fig. 1.7 as the dash-dot curve.

This discussion of reduction in G with increasing shearing strain amplitudes leads to the conclusion that for important installations which may be subjected to dynamic loadings, it is necessary to conduct laboratory tests to evaluate the G/G_o vs γ/γ_r curve for each significant soil layer. However if previously published curves are to be adopted for preliminary studies, it should be noted that recent test data agree better with the Hardin-Drnevich curves than with the Seed-Idriss curves.

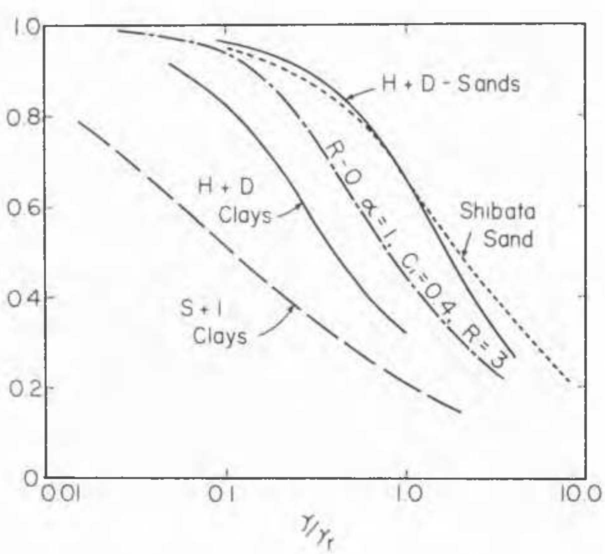

Fig.1.7 Dimensionless Curves for Shear Modulus Reduction with Increasing Shear Strain

A correlation between the numbers of blows per foot (N) in the standard penetration test and the dynamic shear modulus, G_o, was presented by Ohaski and Iwasaki (1973) as $G_o(kg/cm^2) = 120N^{0.8}$. This is an approximate equation representing a least square line fit on a log-log plot of scattered data from a variety of soils. Thus it represents only a first approximation and variations of ± a factor of 3 to 4 might be expected between this expression and data from a particular site.

Correlations between dynamic moduli and shearing strength have been considered by Barkan, et al (1974), for soils in general, Enami and Ohhashi (1973) and Hara, et al (1974) for cohesive soils, and Khazin and Goncharov (1974) for frozen soils. Hara, et al, (1974) first obtained a relation between G_o and N as $G_o(kg/cm^2)=158N^{0.668}$, then another expression relating the shear strength from the unconfined test, S_u, and N as S_u $(kg/cm^2)=0.297N^{0.72}$, and finally obtained $G_o \approx 500\ S_u$ for cohesive soils ($0.50<e<2.5$). They found the average constant of 500 from data in which this value ranged from 250 to 1430. Khazin and Goncharov (1974) presented an empirical equation for the strength of frozen soils in terms of the longitudial wave velocity and damping factor (coefficient of attenuation) as obtained by ultrasonic waves. Possibly this approach could be applied to unfrozen soils, but in terms of shear wave velocity and attenuation of shear waves.

Swiger (1974) presented a comparison of settlements from a large-scale field load test on a cohesionless soil deposit with settlements calculated from soil moduli obtained from cross-hole tests and adjusted to correspond with average strains in the sand. He found good agreement between these results, and recommended adoption of the strain-corrected values of G_o, as obtained from cross-hole tests, for settlement analysis. However, he stressed that corrections for secondary settlement must also be included.

1.6 Conclusions

The low-amplitude values of shear modulus, G_o, can be obtained satisfactorily by seismic tests, particularly by the cross-hole test, or by laboratory resonant column tests which include the secondary time effect. Laboratory tests to determine the shearing strength of soils must either be carried out at a strain rate comparable to that to be encountered in situ, or the static shearing strength must be corrected by a "strain rate factor". The shape of the dynamic stress-strain curve between the initial slope G_o and the ultimate strength τ_{max} is approximately hyperbolic and can be represented in a dimensionless form on a plot of τ/τ_{max} vs γ/γ_r. One curve can represent this relationship for sands and another curve for clays. The reference strain, $\gamma_r=\tau_{max}/G_o$, is a basic parameter in evaluating the reduction of shear modulus when shearing strains exceed about 10^{-6}.

Correlations of dynamic soil properties by field tests are leading to useful results, and continued research on this topic is strongly encouraged.

2. LIQUEFACTION AND CYCLIC DEFORMATION OF SOILS UNDER UNDRAINED CONDITIONS (by Y. Yoshimi)

2.1 Introduction

Dynamic loading may cause both coarse grained and fine grained soils to fail under undrained conditions, as in the case of liquefaction of sands during earthquakes. Although flow of unsaturated soils due to vibration was reported by Mogami and Kubo (1953) and Casagrande (1971), the majority of the researches on the dynamic strength of soils during the last decade has concerned the liquefaction of saturated cohesionless soils.

In a narrow sense of the word, liquefaction means a complete loss of shear strength which can occur when a loose cohesionless soil is subjected to shear stress, either monotonic or cyclic. In a broader sense, the term liquefaction has also been used to denote a partial loss of shear strength due to buildup of pore water pressure, e.g., "partial liquefaction" by Taylor (1948), "initial liquefaction with limited shear strain potential" by Seed et al. (1975a), or "cyclic liquefaction" by Casagrande (1976).

This chapter concerns recent developments in soil liquefaction in the broader sense, with emphasis on laboratory studies based on literature published since the Specialty Conference on Soil Dynamics (1969). Excellent state-of-the-art reports on this subject have recently been presented, e.g., by Faccioli and Reseniz (1975), Seed et al. (1975a), Casagrande (1976), Seed (1976), and by Lee and Focht (1976).

It is convenient for the subsequent discussion to classify the stress conditions causing liquefaction as follows: (1) monotonic shear stress as in a slope under static conditions, (2) completely reversed cyclic shear stress on Element A or B in Fig. 2.1 during earthquakes, and (3) partially reversed or unreversed cyclic shear stress on Element C or D during earthquakes, or Element D near an offshore structure during storms.

2.2 Mechanism of Soil Liquefaction

2.2.1 Liquefaction involving Collapse of Soil Structure

When a saturated cohesionless soil of low to medium density is subjected to a single or repeated application of shear stress, the soil structure may undergo a sudden collapse and become virtually suspended in pore water. When this occurs in a level ground, the effective stress reduces to zero. This so-called quicksand condition can last as long as an upward seepage with the critical hydraulic gradient, $(G - 1)/(1 + e)$, is maintained.

When the collapse of soil structure occurs in a slope, the slope will undergo a nearly unlimited flow, becoming as flat as 3° to 4° from the horizontal (Casagrande, 1971). This type of liquefaction and flow can be simulated in the laboratory by an undrained triaxial compression test in which the deviator stress is applied by dead load. Castro (1969) conducted this type of test on three sands, and obtained for each sand a nearly unique relationship between the void ratio and the effective minor principal stress after failure, regardless of the initial confining stresses. With this "critical void ratio" line, one can differentiate the conditions under which unlimited and limited flow can occur during monotonic loading (Youd, 1973). Because of the sustained shear stress and non-zero angle of shearing resistance, the minor principal stress did not reduce to zero although it bacame as small as 0.016 kg/cm^2 for a very loose sand.

When liquefaction is induced by cyclic shear stresses, the sudden collapse of the soil structure as evidenced by a sudden increase in the shear strain and in the pore water pressure is preceded by a gradual buildup of pore water pressure with negligible shear strain as shown in Fig. 2.2.

The buildup of pore water pressure due to cyclic shear under undrained conditions is attributed to an irreversible change in the soil structure probably involving microscopic slips along intergranular contacts. The tendency of the soil to contract due to cyclic shear is counteracted by a rebound due to the reduction in the effective stress to satisfy the conditions of constant volume and constant total stress (Yagi, 1972; Martin et al., 1975).

The results of undrained cyclic shear tests shown in Fig. 2.2 conducted at several levels of shear stress amplitudes can be summarized as shown in Fig. 2.3. The solid curve in Fig. 2.3(a) shows the shear stress amplitude τ_d plotted against the number of cycles to initial liquefaction N_ℓ, which denotes "a condition where, during the course of cyclic stress applications, the residual pore water pressure on completion of any full stress cycle becomes equal to the applied confining pressure" (Seed et al., 1975a). The term "initial liquefaction" was also used in a broader sense to denote a condition where a sudden increase in the pore pressure or the shear strain was imminent, as indicated by N_p or N_s in Fig. 2.2.

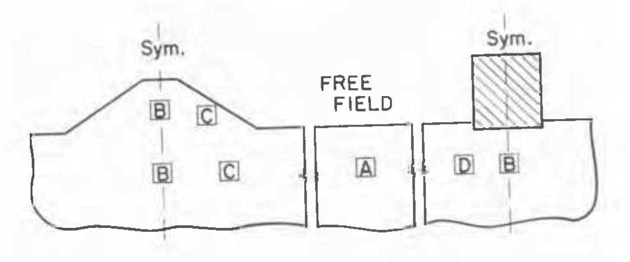

Fig. 2.1 Stress Cinditons Causing Liquefaction

The dashed line in Fig. 2.3(a) can be obtained by plotting the number of cycles to failure, N_f, corresponding to a failure strain, γ_f. In the example shown, the difference between N_ℓ and N_f is small because the strain increases rapidly after the pore pressure becomes equal to the applied confining pressure, $\bar{\sigma}_o$. It is evident in Fig. 2.3 that there is a threshold shear stress amplitude, τ_{cr}, below which the pore pressure does not build up at all.

2.2.2 Undrained Cyclic Deformation with Pore Pressure Buildup

When dense saturated cohesionless soil is subjected to cyclic shear stress at a level somewhat lower than the static strength, the pore water pressure may build up gradually until it reaches the applied confining pressure, or to a condition of initial liquefaction. The pore pressure buildup in a dense cohesionless soil under cyclic loading conditions is due to the fact that even a dense soil is contractive during shear at small strains.

As soon as the shear strain exceeds a certain limit, however, the soil becomes dilative causing a drop in the pore water pressure

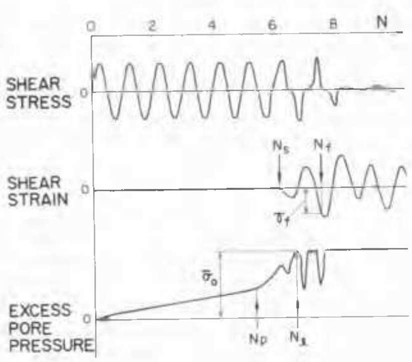

Fig. 2.2 An Example of Undrained Cyclic Simple Shear Test on Loose Saturated Sand

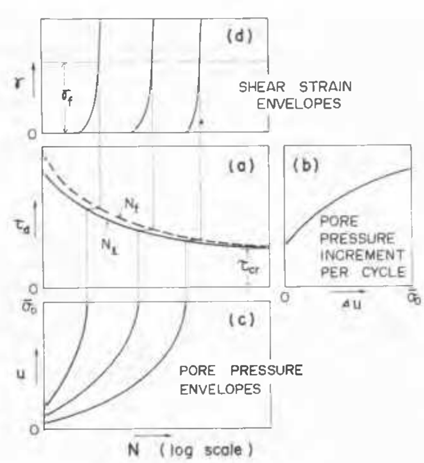

Fig. 2.3 Typical Result of Undrained Cyclic Shear Test on Loose Saturated Sand

with a consequent recovery of the effective stress. As a result, the condition of zero effective stress occurs only momentarily when the shear stress is zero, and the soil retains considerable shear modulus even after the initial liquefaction. In fact, a dense soil cannot be strained beyond a certain limit regardless of the level of shear stress amplitude, provided it stays below the static shear strength. According to De Alba et al. (1976), the limiting shear strain at 10 stress cycles for a uniform fine sand was less than 10 per cent for relative densities above 80 per cent. The above phenomenon has been called "cyclic mobility" by Castro (1975), "initial liquefaction with limited shear strain potential" by De Alba et al. (1976), or "cyclic liquefaction" by Casagrande (1975).

2.3 Factors Influencing Liquefaction of Cohesionless Soils

As far as cohesionless soils are concerned, their resistance to liquefaction appears independent of the frequency of cyclic loading, e.g., from 1 Hz to 12 Hz for a clean sand (Yoshimi and Oh-oka, 1975), and from 1/12 Hz to 1 Hz (Lee and Focht, 1975b), which cover a usual range of frequencies for seimic loading. For a longer range of frequencies, however, the liquefaction resistance increased somewhat with decreasing frequencies (Wong et al., 1975).

2.3.1 Confing Stresses and Boundary Deformation of Specimens

According to cyclic triaxial tests on a clean sand, the cyclic shear stress required to cause initial liquefaction in a given number of stress cycles was nearly proportional to the initial confining pressure between 1 and 15 kg/cm^2 (Lee and Seed, 1967).

In many cases a soil element on the field (Fig. 2.1) is consolidated anisotropically and sheared under plane-strain conditions. Simple shear tests and torsion tests of various forms have been devised to better simulate their field behavior than the triaxial test (see Section 2.8). On the basis of cyclic torsion tests, it has been shown that for a given initial vertical effective stress the resistance to liquefaction increased with an increase in the initial coefficient of earth pressure at rest, K_0 (Ishihara and Li, 1973; Ishibashi and Sherif, 1974; Ishihara and Yasuda, 1975; DeAlba et al., 1976). There are indications that the liquefaction resistance is nearly proportional to the initial average effective stress, $\bar{\sigma}_0=(1+2K_0)\bar{\sigma}_v/3$, in which $\bar{\sigma}_v$ is the vertical effective stress.

A specimen boundary over which confining pressure is applied is either flexible as in the triaxial test, or rigid as in the simple shear test using a metal box. Over the flexible boundary the membrane around a specimen is pushed out as the pore water pressure builds up, causing an overestimation of the resistance to liquefaction. This membrane compliance effect is particularly important in liquefaction tests in which the pressure difference across the membrane undergoes a substantial change.

DeAlba et al.(1976) showed that the liquefaction resistance of a uniform fine sand (Monterey No. 0 sand) determined from their large-scale simple shear tests had to be reduced by 20 to 30 per cent to correct for the membrane compliance effect. On the basis of cyclic triaxial tests on the same sand, Wong et al.(1975) showed that the liquefaction resistance of 12-in.(300-mm) specimens was about 10 per cent lower than 2.8-in.(70-mm) specimens: another indication of a membrane effect. Therefore, we must carefully assess the boundary compliance effect before we attempt to make quantitative statements about liquefaction resistance or compare liquefaction resistances determined with different methods.

2.3.2 Soil Type, Gradation, and Density

Previous records of seismic damage in Japan have revealed that such signs of soil liquefaction as sand boils, mud spouts, and flotation of wooden piles were found in Recent fluvial deposits and uncompacted sandy fills (Kuribayashi and Tatsuoka, 1975). According to Youd and Hoose (1977), Recent deltaic deposits and saturated deposits have also shown high susceptibility to liquefaction, whereas clay-rich and pre-Pleistocene deposits have not generally been affected by liquefaction.

Extensive laboratory liquefaction tests have been conducted during the past decade on clean, uniformly graded fine sands, which appeared least resistant to liquefaction as far as the gradation was concerned (Lee and Fitton, 1969; Seed and Peacock, 1971; Wong et al., 1975).

It has been argued that the apparently superior liquefaction resistance of coarse sands and gravels in the laboratory might be attributed in part to the effect of membrane compliance (Wong et al., 1975). On the other hand, the coarser granular soils should exhibit considerably greater stability in the field because their high permeability would either preclude a full development of pore water pressure or reduce the duration of fully liquefied condition (Wong et al., 1975; Seed and Booker, 1976).

On the basis of cyclic undrained triaxial tests on undisturbed or reconstituted specimens of medium dense to dense sands, it has been shown that the shear stress ratios required to produce a given strain in a given number of cycles at a given relative density varied between wide limits depending on the soil type and gradation (Marcuson and Townsend, 1976; Castro and Poulos, 1976).

Attempts have been made to determine the effect of fines on the liquefaction resistance of sands either with reconstituted specimens or undisturbed specimens. The latter should be preferred because the effect of soil structure (see Section 2.3.3) would be more pronounced as the percentage of fines is increased.

On the basis of a survey of seismic records in Japan, Kishida (1969) reported that well-graded soils were less susceptible to liquefaction than uniformly graded soils. On the other hand, recent laboratory test results show that well-graded soils appear weaker than uniformly graded soils at a given relative density (Wong et al., 1975). It is conceivable that the superior strength of well-graded soils in the field might be attributed to the fact that they tend to be deposited to form a more stable structure.

On the basis of large-scale laboratory tests on a clean, uniformly graded fine sand (Monterey 0 sand), De Alba et al. (1976) showed that the stress ratio causing initial liquefaction in 10 to 30 stress cycles was nearly proportional to the relative density up to about D_r = 75 per cent. However, for sands in-situ which often contain some silt and clay, the relative density may not be a good measure for expressing the resistance to liquefaction (Castro, 1975).

The liquefaction resistance of non-plastic rock flour soils with the mean grain size of 0.03 mm to 0.05 mm were found to depend on aging after consolidation (Donovan and Singh, 1976). A volcanic cohesionless soil locally called "Shirasu" in Japan was reported significantly less resistant to liquefaction than fine sands of similar gradation (Yamanouchi et al., 1976).

2.3.3 Specimen Preparation, Strain History, and Soil Structure

On the basis of extensive cyclic undrained tests on reconstituted specimens of sands, it has been clearly shown that the liquefaction and cyclic deformation characteristics of sands were markedly influenced by the method of preparing the test specimens (Ladd, 1974, 1976; Mulilis et al., 1975; Marcuson and Townsend, 1976). For both medium dense and dense specimens of sand, the specimens prepared by compacting moist sand showed significantly greater resistance to cyclic undrained shear than those compacted dry, the difference being as much as 110 per cent.

Recent laboratory test results on undisturbed samples of sands have indicated that the liquefaction resistance of the undisturbed specimens were generally higher than that of the specimens which were reconstituted to the same density, e.g., up to 45 per cent higher than the specimens compacted by moist tamping (Mulilis et al., 1975), and 65 to 112 per cent higher than the specimens compacted by dry tamping (Marcuson and Townsend, 1976). For sands containing 21 to 32 per cent fines, the liquefaction resistance of the undisturbed specimens was 6 to 26 per cent higher than the specimens remolded and consolidated to the same density (Watanabe et al., 1975). More studies of this type using truly undisturbed samples are needed to establish quantitative relationships between the undisturbed and remolded strengths.

On the basis of cyclic undrained tests on sands it has been shown that the specimens which had been subjected to cyclic stresses at moderate levels (presheared specimens) exhibited considerably higher resistance to liquefaction (Finn et al., 1970; Bjerrum, 1973; Seed et al., 1975a). For example, the liquefaction resistance of a fine sand increased nearly 50 per cent as a result of preshearing, although the preshearing caused only a minor increase in the relative density, i.e., from 54.0 per cent to 54.7 per cent (Seed et al., 1975d).

The effect of preshearing is an important factor to be considered when we attempt to evaluate the liquefaction resistance of in-situ soil subjected to seismic or ocean wave loading. In the case of foundation soil below offshore structures subjected to wave forces, a train of cyclic stresses during a storm may increase the dynamic bearing capacity for subsequent storms (Lee and Focht, 1975a). Natural deposits of soils in seismically active regions must have been presheared by a number of earthquakes. It is conceivable that such natural preshearing might account for at least a part of the increased liquefaction resistance of undisturbed samples compared to reconstituted samples.

It seems reasonable to assume that both the effect of sample preparation and preshearing may be attributed to soil structure. Some attempts have been made to explain the effect of sample preparation on the basis of the orientation of intergranular contact surfaces and the electrical conductivity of the specimens saturated with an electrolytic solution. According to Mulilis et al.(1975), the smaller the electrical conductivity at a given density, the greater was the liquefaction resistance. However, the effect of preshearing and the difference between the undisturbed and remolded strengths have not yet been fully explained on the basis of soil structure.

In marked contrast to the beneficial effect of preshearing at moderate strain levels, severe shear strains including liquefaction have detrimental effects on the undrained shear strength, i.e., saturated sand which has been liquefied and then reconsolidated may exhibit much smaller resistance to subsequent applications of cyclic shear stresses despite the fact that the sand has become denser during the reconsolidation process. This reliquefaction phenomenon was observed in cyclic triaxial and simple shear tests as well as in shaking table tests on bulk samples of sand in a large container (Finn et al., 1970; Finn, 1972).

The reduced resistance of preliquefied sand has been attributed to the development of loose zones in the specimen which would govern the resistance during the subsequent stress applications (Emery et al., 1973; Castro, 1975). However, cyclic triaxial tests by Mulilis et al. (1975) show that the uneven density distribution alone cannot account for the drastic reduction in the liquefaction resistance. Lee (1976) pointed out that the change in soil structure caused by liquefaction as evidenced by drastically increased compressibility (Lee and Albaisa, 1974; Yoshimi et al., 1975) would account for the reduced resistance to subsequent shear stress pulses. From the point of view of soil structure, we may argue that the condition following complete liquefaction represents a virgin state which is less stable than the condition prior to the first liquefaction which has been stabilized by preshearing of a sort.

Whether or not the reliquefaction phenomenon occurs in the field is important when we attempt to evaluate the liquefaction potential of a place like Niigata, Japan, for future earthquakes. Lee (1976) argued that "small cyclic stresses which develop in the ground during aftershocks of the main earthquake are sufficient to restabilize a once liquefied, reconsolidated soil." Moderate earthquakes which occur between strong ones may also contribute to restoring the resistance by preshearing.

2.4 Liquefaction of Level Ground during Earthquakes

It is recognized that the progressive reduction in effective stresses leading to liquefaction during earthquakes is governed by cyclic shear stresses which in turn is due primarily to shear waves propagating upwards from the bedrock.
The methods which have been proposed to evaluate the liquefaction potential of level ground may be classified as follows: (1) empirical criteria of liquefaction potential based on field observations during earthquakes or on dynamic tests in the field; (2) comparison of computed shear stresses in the field with liquefaction resistance determined in the laboratory; and (3) prediction of liquefaction in the field by analyses based on mechanical models of soil elements.

2.4.1 Empirical Criteria of Liquefaction Potential

Extensive liquefaction occurred in the level sandy ground in Niigata, Japan, during the Niigata earthquake of 1964 (magnitude = 7.5, epicentral distance ≑ 55 km), and caused settlement and tilting of more than 200 reinforced concrete buildings, tilting of bridge piers, large displacements of underground structures, and severe damage to life lines in general. Detailed descriptions on the liquefaction damage were presented by the Japanese Society of Soil Mechanics and Foundation Engineering (Soil and Foundation, 1966), and excellent summaries were given by Seed and Idriss (1967) and Seed (1970).

By comparing the standard penetration blow counts before and after the earthquake, Koizumi (1966) proposed a "critical blow count, N_{cr}," as shown in the solid line in Fig. 2.4, on the basis of the hypothesis that the soil that had liquefied should have experienced an increase in its blow count. Thus, the blow count before the earthquake that had fallen on the left side of N_{cr}-curve increased as a result of the earthquake, and those on the right side decreased.

Kishida (1966) studied the relationship among the degree of damage, the depth to and the standard penetration blow count at the lowest point of foundation for 185 reinforced concrete buildings located in the most heavily damaged area in Niigata. Forty out of 63 buildings (64 per cent) on shallow foundations and 49 out of 75 buildings (65 per cent) on short piles settled more than 50 cm and/or tilted more than 1.0 degree. The lower ends of the piles reached depths between 5 m and 10 m where the blow count was less than 15. Kishida then plotted the blow count at the bottom of shallow foundations or at the pile tips, and showed that the data points could be separated into two groups by the dashed lines as shown in Fig. 2.4, i.e., those on the left side suffered heavy damage (more than 50 cm in settlement or 1.0° in tilting), and those on the right lighter damage. The effective overburden pressure on the right side scale has been computed for a saturated soil density of 1.90 t/m^3 and a depth of the ground water table of 1.0 m.

Even in the liquefied area wooden houses suffered relatively minor damage. This probably explains why liquefaction-induced damage to buildings had not been conspicuous in old records of earthquakes in Japan where heavy structures were very few. However, old documents on earthquakes contain numerous references to sand boils, mud spouts, and flotation of buried wooden piles, indicating occurrences of liquefaction.

Recent reviews of these documents show that the signs of liquefaction were observed at a

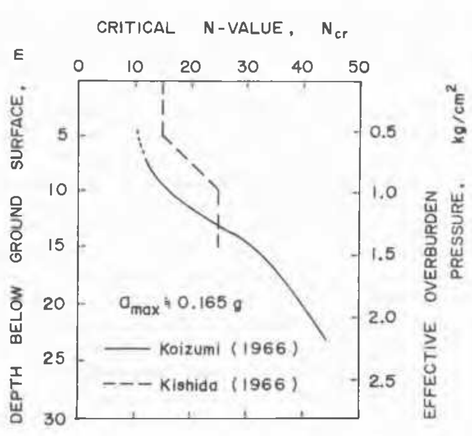

Fig. 2.4 Empirical Criteria of Liquefaction for the Niigata Earthquake of 1964

number of alluvial deposits and reclaimed sites throughout Japan (Kishida, 1969; Kuribayashi et al., 1974; Kuribayashi and Tatsuoka, 1975). For a given magnitude, M, there appears to be a limiting epicentral distance, R, beyond which liquefaction was unlikely to occur. The limiting distance R in km for $M > 6.0$ may be expressed as follows (Kuribayashi and Tatsuoka, 1975):

$$\log R = 0.77\,M - 3.6 \qquad (2.1)$$

In order to compare field observations concerning liquefaction and non-liquefaction during different earthquakes, the cyclic shear stress ratio, $\tau_d/\overline{\sigma}_o$, has been plotted against the relative density (Seed and Peacock, 1971; Whitman, 1971), or against the standard penetration blow count corrected for a certain effective overburden stress (Castro, 1975; Seed et al., 1975d). The curve in Fig. 2.5 gives the lower bound for the stress ratios causing liquefaction based on observations at 38 sites during 14 earthquakes, and is expected to give a reasonable criterion for Magnitude 7.5 earthquakes, but a concervative estimate for earthquakes of smaller magnitudes (Seed, 1976).

Attempts have recently been made to apply statistical methods to treating field data to discriminate between liquefiable and non-liquefiable conditions (Christian and Swiger, 1975; Tanimoto and Noda, 1976; Yegian, 1976). Such approaches are attractive in view of the probabilistic nature of earthquakes. But the reliability of the methods is still limited by the current lack of good field data during strong earthquakes.

Florin and Ivanov (1961) described a method by which the susceptibility of soil to liquefaction could be predicted on the basis of ground subsidence caused by blast tests. Kummeneje and Eide (1961) and Prakash and Gupta (1970) have conducted similar tests in which pore pressure measurements were made below the ground surface. Ishihara and Mitsui (1972) measured dynamic pore water pressure and vertical acceleration in saturated sand near a pile while it was vibrated vertically. They found that the relationship between the pore pressure and acceleration for uncompacted sand was markedly different from that for compacted sand, the difference being more pronounced than the difference in the standard penetration blow counts would indicate.

The in-situ tests described above may be used to compare the liquefaction potential at a proposed site with that at a site where liquefaction resistance is known.

2.4.2 Comparison of Computed Shear Stresses with Liquefaction Resistance determined in the Laboratory

Seed and Idriss (1971) proposed a practical method for evaluating liquefaction potential of horizontal deposits of cohesionless soils on the basis of the seismicity of the site under consideration (the magnitude of the earthquake and the maximum ground surface acceleration) and of the soil conditions (the grain size, the depth of the ground water table, and the standard penetration blow counts).

This method was successfully applied to 35 cases of which liquefaction occurred in 23 cases. The authors cautioned, however, that the method involved a number of assumptions, and that it should be regarded as an approximate method by which to extend previous field observations to new situations. One of these assumptions concerns conversion of a train of irregular shear stress pulses during an actual earthquake to a certain number of equivalent uniform shear stress pulses having an amplitude of 65 per cent of the maximum shear stress in the irregular time histories. The number of cycles were assumed to increase with the magnitude of the earthquake, i.e., 10, 20, and 30, for magnitudes 7, 7.5, and 8, respectively.

The use of the equivalent uniform shear stress was necessary because the liquefaction resistance of soils had been determined from cyclic tests of uniform shear stress amplitudes. Various studies have since been made to evaluate the effect of irregular stress pulses on liquefaction. Methods based on a generalized equivalent uniform cycle concept or a cumulative damage concept have been described and compared (Lee and Chan, 1972; Annaki and Lee, 1976; Seed, 1976; Valera and Donovan, 1976) with the conclusion that the methods had a relatively minor effect on the final result.

Ishihara and Yasuda (1975) have employed a different method in which specimens of sand were actually subjected to irregular stress pulses simulating time histories recorded during earthquakes. The equivalent uniform stress amplitudes determined from their test results were somewhat lower than the value proposed by Seed and Idriss (1971) for Magnitude 7.5 earthquakes, but higher for Magnitude 7 earthquakes.

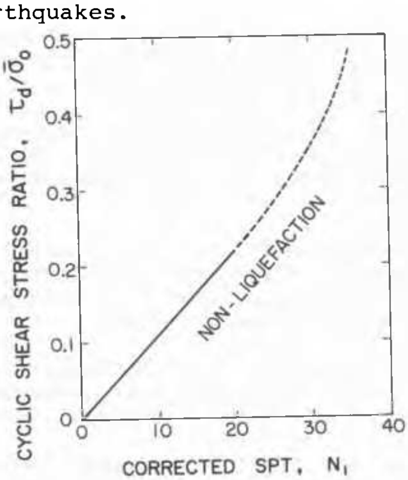

Fig. 2.5 Lower Bound for Shear Stress Ratios Causing Liquefaction (Seed et al, 1975d)

In the simplified procedure by Seed and Idriss (1971), the maximum dynamic shear stress and the liquefaction resistance are determined from simple formulas and charts which have been prepared for representative cases. For more detailed studies for particular sites, the maximum dynamic shear stress is determined by conducting dynamic response analyses (Schnabel et al, 1972), and the liquefaction resistance by appropriate laboratory tests on soil samples taken from the sites.

On the basis of multi-directional shaking tests on dry sand, it was inferred that its liquefaction resistance would be about 10 per cent lower than that in uni-directional, completely reversed cyclic shear (Pyke et al, 1975; Seed et al, 1975a).

In order to estimate the liquefaction resistance in the field, Seed (1976) recommended that the laboratory test data be corrected as follows: (1) the shear stress ratio causing liquefaction in cyclic simple shear tests should be reduced by 10 per cent to allow for multi-directional shaking in the field; and (2) the shear stress ratio causing liquefaction in cyclic triaxial tests (one-half the dynamic deviator stress divided by the consolidation pressure) should be multiplied by 0.57 if the coefficient of earth pressure at rest in the field, $K_o = 0.4$, and by 0.9 to 1 if $K_o = 1$, to correct for the plane strain condition and multi-directional shaking in the field.

According to a questionnaire survey conducted by the writer from May to August, 1976, the simplified procedure by Seed and Idriss was widely used by practicing engineers in the U.S.A. and Japan, mostly as a preliminary step towards more detailed studies involving dynamic response analyses and laboratory tests.

2.4.3 Analytical Studies of Development and Dissipation of Pore Water Pressure

Attempts have been made to predict the development of pore water pressure during undrained cyclic shear on the basis of constitutive relations of soil elements. In most of these studies the pore pressure increment per stress cycle, Δu, is expressed in terms of the shear stress amplitude or dilatancy characteristics of soils determined experimentally.

Shibata et al (1972) proposed a parabolic relationship between Δu and τ_d as shown in Fig. 2.3(b), and used it to estimate pore pressure development due to irregular shear stress pulses. Yagi (1972) derived a Δu vs. τ_d relationship from more fundamental empirical relationships among the volumetric strain, shear stress ratio, and rebound characteristics of dry sand. Similar studies have been made by Martin et al (1975), Oh-oka (1976), Finn et al (1976b), and Liou et al (1976).

Finn et al (1976b) showed that their constitutive relations could successfully simulate nonlinear stress-strain curves during cyclic shear as well as the relationship between the liquefaction resistance and the number of cycles to liquefaction for a soil element (Fig. 2.3(a)). With their mechanical models of soil elements they could conduct nonlinear effective stress analyses of dynamic response of horizontal saturated sand deposits during earthquakes (Finn et al., 1976).

Ishihara et al. (1975) proposed a unique method in which pore water pressure and shear strain during undrained cyclic shear of irregular time histories were assumed to accumulate only when the ratio of the shear stress amplitude to the current mean principal stress exceeded the previous peak. The method was then combined with dynamic response analyses to study the development of pore water pressure in horizontal deposits of saturated sand during actual earthquakes, and the computed results agreed reasonably well with the observed field behavior (Ishihara et al., 1976).

According to the eyewitness accounts during the Niigata earthquake of 1964, the sand boils or the settlement of the buildings began to take place some time after the ground shaking had stopped. In the case of a two-story reinforced concrete building at the Niigata airport which settled about 1 m, the time interval between the first major shock and the settlement was estimated to be about 40 sec by reenacting the scene of evacuation from the office upstairs.

The time lag probably indicates that the soil at some depth was first liquefied during the earthquake, and that the excess pore water pressure in the liquefied zone caused an upward seepage through the surface soil and subsequent loss in the bearing capacity. This transient seepage problem was analyzed by applying the Terzaghi consolidation theory by Ambraseys and Sarma (1969), and Yoshimi and Kuwabara (1973). A more recent study by Yoshimi et al. (1975) using soil properties determined from large-scale consolidation tests of liquefied sand shows that the maximum pore pressure in the layer overlying the liquefied layer is primarily governed by the ratio of the coefficients of permeability of the two layers.

Numerical analyses have been made to estimate the time histories of pore water pressure in horizontal deposits of saturated sand, taking into account both development and dissipation of pore water pressure. To determine dynamic stresses, Seed et al. (1975c), and Seed and Booker (1976) used soil properties based on the initial effective stress, whereas Finn et al. (1976b) updated the effective stress by feeding back the current pore pressures. Liou et al. (1976) used a different approach based on the method of characteristics (Streeter et al., 1974). It appears that these analytical methods have achieved an adequate level of sophistication, considering the uncertainties in input earthquake motions and properties of soil deposits in situ.

2.4.4 Settlement due to Liquefaction

Following liquefaction, a horizontal layer of saturated sand settles as it consolidates under its own weight. On the basis of cyclic triaxial tests on uniformly graded sands, Lee and Albaisa (1974) estimated that volumetric strains during the post-liquefaction settlement would be from 1 % to 4 %, which agreed with field observations, shaking table test results, and with the results of large-scale one-dimensional consolidation tests by Yoshimi et al. (1975). However, higher strains could occur if vibration was continued beyond liquefaction or if the sand had been unusually loose.

2.5 Liquefaction of Slopes, Embankments and Level Ground below Structures

2.5.1 Liquefaction of Slopes and Embankments

On the basis of field observations and analytical studies, it has been inferred that a number of failures and deformations of slopes and embankments during earthquakes could be attributed to liquefaction or cyclic liquefaction of cohesionless soil, either comprising the slopes or included as lenses.

Seed (1968) presented a comprehensive review of landslides during 37 earthquakes due to soil liquefaction beginning with the Helice earthquake of 373 BC. The slope failures observed during these and more recent earthquakes, e.g., the Tokachioki earthquake of 1968 (Ikehara, 1970) and the San Fernando earthquake of 1971 (Seed et al., 1975b), have been classified into the following types:

(1) Flow slides due to liquefaction of cohesionless soils comprising the slopes; (2) slope failures due to liquefaction of thin layers of sand; (3) slope failures of predominantly cohesive soils due to liquefaction of sand lenses; (4) slumping of embankments on firm foundation due to cyclic deformation. In addition, many failures of earth retaining structures have been reported due to liquefaction of the backfill.

The presence of a number of hydraulically filled earth dams in seismically active regions in the U.S.A. has prompted intensive efforts to evaluate their stability during earthquakes. Seed and his colleagues developed a procedure to evaluate the overall deformation and stability of embankment cross-sections by comparing computed dynamic stresses with dynamic strengths of soil elements determined in the laboratory.

Applications of the procedure to the Sheffield Dam during the Santa Barbara earthquake of 1925 (Seed et al., 1969) and to the San Fernando Dams during the San Fernando earthquake of 1971 (Seed et al., 1975b) led to the conclusion that the slides were caused primarily by liquefaciton of loose saturated cohesionless soils comprising the embankments. The conclusion seemed to be supported by field observations made after the slides.

The above procedure has been applied to the existing earth dams in the U.S.A.to evaluate their stability during earthquakes, e.g., by Marcuson and Krintzsky (1976).

It is interesting to note that a sudden rise in pore water pressure was actually observed during the Tokachioki earthquake of 1968 below a railway embankment near Misawa, Japan (Ikehara, 1970), and in the upper San Fernando Dam during the San Fernando earthquake of 1971 (Seed et al., 1975b).

On the basis of detailed field observations, dynamic response analyses and laboratory tests, Seed (1968) demonstrated that the slides during the Alaskan earthquake of 1964 were initiated by liquefaction of seams or lenses of saturated sandy soils. In the process he enlightened the profession by illustrating the importance of minor geologic details in determining the occurrence and characteristics of landslides during earthquakes due to soil liquefaction.

Slumping of embankments are quite common during earthquakes because even dense cohesionless soils may be affected. Youd (1973) identified this type of failure as lateral-spreading landslides caused by cyclic liquefaction, and recognized the possibility that migration of pore water into a zone of soil which had dilated during cyclic shear could cause loosening, leaving it in a condition vulnerable to subsequent stress applications.

2.5.2 Liquefaction of Level Ground below Structures

On the basis of analyses, laboratory cyclic shear tests, shaking table tests and field observations, it has been shown that the liquefaction potential of saturated sand directly below a heavy structure such as a multistory reinforced concrete building was considerably smaller than the same sand in the free field (Yoshimi and Oh-oka, 1975; Ishihara and Matsumoto, 1975; Yoshimi and Tokimatsu, 1977).

In Fig. 2.6 the ratio of the average settlement, S_a, to the depth of liqefaction, D, for 43 reinforced concrete buildings in Niigata, Japan, is plotted against the ratio of the width of the buildings to the depth of liquefaction, B/D. The depth of liquefaction was estimated by the simplified procedure by Seed and Idriss (1971). Also shown in the figure are the results of shaking table tests by Yoshimi and Tokimatsu (1977). Both the field observations and the laboratory tests show the same trend that the greater the width ratio, the smaller the settlement ratio. This may be attributed to the fact that the liquefaction potential directly below the structure is considerably smaller than that away from the structure, primarily on account of a confining effect by the structure. With further research, this may lead to economical measures to minimize liquefaction damage to a wide structure such as oil storage tanks.

Some of the methods for evaluating the liquefaction potential of horizontal deposits of cohesionless soils during earthquakes as described in Section 2.4.2 have been applied to an investigation of the stability of dense sand below an offshore oil storage tank subjected to ocean wave loading (Lee and Focht, 1975). Compared to seismic loading, ocean wave loads have longer duration, longer wave periods, and more frequent occurrences of moderate storms. The second and third items are expected to have beneficial effects in allowing partial drainage and preshearing, respectively.

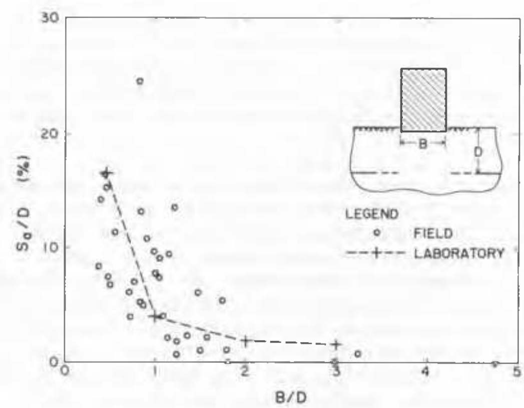

Fig. 2.6 Influence of Width Ratio on Settlement of Structures

2.6 Remedial Measures to Prevent Damage due to Liquefaction of Cohesionless Soils

When a proposed site is judged susceptible to liquefaction, we may recommend relocation of the site, stabilization of the proposed site, or pile foundation.

Densification of loose cohesionless soils by vibroflotation and a variety of deep compaction techniques have been employed to prevent damage due to liquefaction. Beneficial effects of vibroflotation were evidenced during the Niigata earthquake of 1964 (Watanabe, 1966), and the Tokachioki earthquake of 1968 (Ohsaki, 1970). Deep vibratory densification methods of various forms have been used extensively in Japan to compact sandy hydraulic fills along the sea coast, e.g., a total area of 3,080 km^2 or a total length of 9,200 km was compacted at five industrial sites from 1961 to 1976, to depths from 8 m to 25 m (Ueda, 1976). In order to meet very severe compaction requirements, we may have to resort to recompaction of excavated soil after temporary dewatering.

Beneficial effects of coarse backfill on preventing pore pressure buildup or reducing the duration of liquefaction were pointed out by Yoshimi and Kuwabara (1973), and confirmed by Yamanouchi et al. (1976) in field vibration tests in which pore water pressure due to driving a steel pipe pile was measured. On the basis of numerical analysis of development and dissipation of pore water pressure, Seed and Booker (1976) showed the possibility that gravel drains would effectively reduce the liquefaction potential of the surrounding sand.

Lowering the ground water table tends to reduce the potential damage due to liquefaction for the following two reasons: (1) It increases the liquefaction resistance of the saturated part of the soil by increasing the effective stress; and (2) It increases the thickness of the unsaturated part of the soil. The depth to ground water table may be increased either by permanent dewatering or by placing a fill. The effect of the latter was demonstrated by Seed and Idriss (1967) concerning the Niigata earthquake of 1964.

Yoshimi and Tokimatsu (1977) showed in their shaking table tests that rigid walls embedded around a structure had a considerable effect on reducing the excess pore pressure below the structure and a marked effect on reducing the settlement of the structure. This method will be particularly advantageous for existing sturctures.

When piles are recommended to penetrate liquefiable soil deposits, we must take into account the loss of lateral resistance and frictional resistance in assessing their bearing capacity and deformation.

2.7 Strength of Saturated Cohesive Soils under Cyclic Loading Conditions

The dynamic strength of compacted clays has long been studied in connection with subgrade design, and a brief review of the effects of strain rate and repeated loading is presented in Section 1.3 in this report. It has been recognized that driving piles into saturated clays could cause a considerable increase in the pore water pressures in the clays, sometimes exceeding the total overburden pressure, and a significant reduction in the undrained shear strength (Orrje and Broms, 1967). In this section the cyclic shear strength of saturated cohesive soils is compared with liquefaction of saturated cohesionless soils.

Although both cohesive and cohesionless soils exhibit similar low-cycle fatigue behavior as shown in Fig. 2.3(a), cohesive soils do not seem to show a sudden collapse of soil structure accompanied by a sudden rise of pore water pressure which characterizes liquefaction of loose saturated sands (Lee and Focht, 1976). Because of technical difficulties in accurate measurements of dynamic pore water pressures in cohesive soils, relatively few data have been reported on their effective stress response (Sangrey et al., 1969; Wilson and Greenwood, 1974). It has been indicated that the pore pressures did not quite reach the initial effective stress even after the effective stress paths touched the failure envelope. Thus, failure must be defined on the basis of shear strains. Lee and Focht (1976) compiled cyclic strength data on 24 cohesive soils, and plotted the ratio of the cyclic strength to the static undrained strength against the number of cycles to failure. For example, cyclic stress ratios from 0.38 to 1.00 were required to cause failure in 10 cycles, and from 0.15 to 0.63 in 100 cycles.

Unlike cohesionless soils whose liquefaction resistance is practically independent of the frequency of cyclic loading (Section 2.3), the cyclic strength of cohesive soils is significantly reduced as the frequency becomes lower (Thiers and Seed, 1969; Arango and Seed, 1974). This plus the fact that square stress pulses cause lower cyclic strength than triangular stress pulses at a given frequency may be attributed to creep behavior of cohesive soils (Lee and Focht, 1976).

More research will be required to study liquefaction of cohesive soils of low plasticity containing significant amounts of sands and silts.

2.8 Experimental Methods Concerning Soil Liquefaction

During the last decade a variety of laboratory testing methods to study soil liquefaction was developed in rapid succession. In this section these methods are described with comments on their advangages and limitations.

2.8.1 Triaxial Tests for Completely Reversed Cyclic Shear Stresses

In order to simulate completely reversed cyclic shear stresses, a specimen is first consolidated isotropically, and then alternating compression and extension tests are conducted by applying cyclic deviator stress $\pm\ \sigma_d$ with the chamber pressure held constant (Seed and Lee, 1966), or the axial stress and the chamber pressure are cycled with a 180° phase angle, i.e., with the sum of the maximum and minimum principal stresses held constant (Shibata et al., 1972). In either method the shear stress and the effective stress on a plane making a 45° angle with the axis of the specimen is expected to simulate the stresses acting on the horizontal plane of Elements A or B in Fig. 2.1. The advantages and limitations of the cyclic triaxial test may be summarized as follows:

<u>Advantages</u>: (1) The cylindrical surface of the specimen which constitutes a major part of the boundary surfaces is free from shear stresses, and potential failure planes are away from the boundaries; (2) the shape and the size of the specimen is suitable for testing undisturbed samples; and (3) it is easy to maintain a constant volume condition and to apply back pressures to saturate the specimen.

<u>Limitations</u>: (1) Only isotropic consolidation can be simulated; (2) the plane strain condi-

tion in the field cannot be simulated; (3) the specimen may show asymmetrical responses to compression and extension; and (4) the specimen may neck and bulge at the top thereby causing a redistribution of density within the specimen, and obscuring quantitative significance of the measured strains (Castro, 1975; Casagrande, 1976).

The cyclic triaxial test has contributed a great deal to quantitative studies of the conditions causing liquefaction (Lee and Seed, 1967; Tanimoto, 1967; Lee, 1976a), and will continue to provide a useful and practical means to the profession, particularly with truly undisturbed soil samples.

Recent studies (Silver et al, 1976) show that consistent results could be obtained by eight different laboratories, provided that details of test procedure are carefully controlled, e.g., the preparation of specimens, the determination of specimen density, and the shape of the shear stress pulses.

2.8.2 Triaxial Tests for Partially Reversed or Unreversed Cyclic Shear Tests

Partially reversed or unreversed cyclic shear tests can be performed on a triaxial specimen by applying symmetrical deviator stress pulses, $\pm \sigma_d$, on an anisotropically consolidated specimen (Huang, 1961; Seed and Lee, 1969), or by applying asymmetrical pulses on an isotropically consolidated specimen.

Huang (1961) vibrated vertically a triaxial cell on which a constant deviator stress was applied with a weight. He concluded correctly that "the key to the solution of the liquefaction problem is to find out the relationship connecting the developed pore pressure, the density of the sand, the intensity of the dynamic action and the state or stress of the sand mass," although he did not report the effect of the duration of vibration or the number of cycles of shear stress pulses.

2.8.3 Cyclic Simple Shear Tests

Simple shear tests of various forms have been conducted in order to simulate more closely than the triaxial test the stress and strain conditions of an element of soil in the field during earthquakes (see 2.3.1).

In earlier tests a rectangular specimen enclosed in six metal plates was sheared by moving either the top or the bottom plate back and forth (Peacock and Seed, 1968; Finn et al, 1970). A disk shaped specimen placed in a wire-reinforced rubber membrane or a stack of thin rings has also been tested in simple shear (Hara et al, 1975). The advantages and limitations of these devices may be summarized as follows:

Advantages: (1) Fully reversed cyclic shear stress can be applied to anisotropically consolidated specimens; and (2) plane strain conditions can be obtained.

Limitations: (1) Because complementary shear stress cannot be applied the stress distribution within the specimen is not uniform; (2) it is difficult to prevent slippage along the top and bottom plates without resorting to fins which may cause local disturbance; and (3) sharp corners of a rectangular specimen makes it difficult to prepare a uniform specimen for undrained tests.

For an instability phenomenon such as the liquefaction of loose sand where failure initiated at a local flaw can propagate quickly through the whole specimen, the inability to ensure uniformity of stresses and strains is likely to cause premature failure.

In order to overcome the above difficulties De Alba et al (1976) developed a unique device featuring a large length-to-thickness ratio of 22.5 and sloped edges away from rigid boundaries. The cyclic shear stress was applied by the inertia of the reaction mass placed on the specimen when the assembly was vibrated horizontally on the shaking table. Because of the large surface area in contact with the rubber membrane, it was found that the membrane compliance effect was significant and had to be corrected (see 2.3.1).

Casagrande (1976) described a unique gyratory apparatus in which the top and bottom of a disk-shaped specimens were subjected to eccentric rotation.

2.8.4 Cyclic Torsion Tests on Hollow Cylinders

Cyclic torsion tests have been conducted on hollow cylindrical specimens of sands (Yoshimi and Oh-oka, 1973; Ishibashi and Sherif, 1974; Ishihara and Yasuda, 1975). The advantages and limitations of the cyclic torsion test may be summarized as follows:

Advantages: (1) Unlike the simple shear tests on short specimens there is no problem concerning the complementary shear stresses because a hollow cylinder is endless in the circumferential direction; (2) fully reversed cyclic shear stress can be applied on anisotropically consolidated specimens; (3) a nearly plane-strain condition can be obtained by preventing the radial strain; and (4) the lateral pressure and therefore K_0 can be measured under certain conditions.

In an apparatus with a provision to measure the lateral stress, it is not easy to maintain the condition of zero radial strain throughout the specimen during consolidation and cyclic shear. But once it has been accomplished, the data give us the average normal effective stress which is useful for establishing fundamental stress-strain relationships. On the other hand, from the point of view of simulating the field conditions as closely as possible, we may prefer to concentrate on maintaining the zero lateral strain condition, sacrificing the determination of the lateral stress. In many practical applications we must work with vertical stresses alone, because reliable estimates of lateral stresses in situ are not readily available, particularly with compacted sands.

Limitations: (1) Undisturbed samples of sand cannot be readily accommodated; and (2) it is difficult to prevent slippage along the top and bottom boundaries without resorting to fins which may cause local disturbance.

There are two alternatives for improving the uniformity of shear strain which is proportional to the radius: reduce the ratio of the wall thickness to the radius, or make the height proportional to the radius. Although more attractive from a practical point of view, the second alternative involves uneven vertical strains during consolidation and unwanted shear strain component in horizontal cross-sections (Yoshimi and Oh-oka, 1975).

2.8.5 Liquefaction of Bulk Samples of Sand in Containers by Vibration or Impact

Loose saturated sand in a container provides an expedient means to demonstrate the occurrence of liquefaction due to vibration or impact, and subsequent consolidation and settlement.

Florin and Ivanov (1961) applied horizontal vibration or impact to a rectangular box containing saturated sand with free surface or with a pervious surcharge. Similar tests were reported by O-hara (1963), Sunami (1965), Tanimoto (1967), and Whitman (1970).

The vibration tests on sand with free surface described above may be regarded as model tests which simulate level ground during earthquakes. Various methods have been tried to simulate free field ground motions, e.g., by employing a large length to height ratio, hinged end walls, foam rubber cushions at both ends, or sloping ground surface at both ends. As far as the writer is aware, the largest length to height ratio was 10.3 (Finn et al, 1972), and the largest specimens were 9.0 m long, 4.0 m wide, and 1.5 m high (Kubo et al, 1975).

Several attempts have been made to apply an impermeable surcharge on the surface of saturated sand in a container in order to increase the initial effective stress (Yoshimi, 1967; Finn et al, 1972; O-hara, 1972). These studies showed clearly that the specimens experienced a gradual buildup of pore pressure prior to liquefaction, in a similar manner to the cyclic triaxial test or simple shear test.

In the light of recent developments in dynamic response analyses of saturated soil deposits described in Section 2.4.3, we can now use shaking table tests to verify our analytical procedures, thereby eliminating the need to establish similitude relations rigorously. This is demonstrated in Fig. 2.7 in which observed excess pore pressures in a layer of model ground of fine sand during horizontal vibration are compared with computed values. The method of analysis was similar to that proposed by Seed et al (1975c), except that the coefficient of volume change which had been evaluated from large-scale consolidation tests (Yoshimi et al, 1975) was assumed to vary with the effective stress.

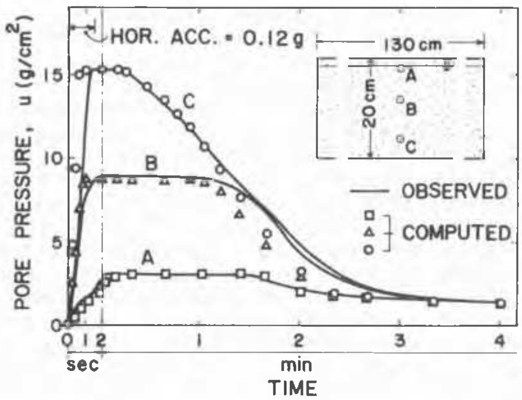

Fig. 2.7 Observed and Computed Pore Pressures for a Shaking Table Test

2.9 Conclusions

Liquefaction of saturated cohesionless soils during earthquakes has been recognized as a temporary loss of shear strength involving pore pressure buildup under undrained conditions. Unlimited flow accompanied by collapse of soil sturcture can occur in saturated cohesionless soils of low to medium densities, whereas there are upper limits for cyclic shear strains for dense cohesionless soils.

Some of the methods proposed to evaluate the liquefaction potential during earthquakes are being employed by practicing engineers to solve geotechnical problems in seismically active regions or in the design of offshore facilities subjected to ocean wave loads.

Among many factors which influence the liquefaction resistance, the confining stress, the soil structure (including density), and strain history seem most important. More research will be required to evaluate the liquefaction resistance of sands containing fines, for which high quality undisturbed samples will have to be used.

2.10 Acknowledgments

The writer is grateful to those who have furnished him valuable information on soil liquefaction. Particular thanks are due to the members of the Committee on Soil Dynamics, the Japanese Society of Soil Mechanics and Foundation Engineering, to the Planning Subcommittee for ASCE National Convention of the Soil Dynamics Committee, and to those who kindly responded to the writer's questionnaire on liquefaction.

3. SEISMIC RESPONSE OF SOIL DEPOSITS, EMBANKMENTS, DAMS AND STRUCTURES (by S. Prakash)

3.1 Introduction

In this chapter, seismic response of soil deposits, slopes, banks, embankments, dams and earth retaining structures shall be discussed. The question of seismic response and stability of earth dams has always been in the minds of those responsible for their safety. The stability of banks and slopes during earthquakes has not been attended to except in the past decade like the "post failure" analysis of slides during the Alaskan earthquake of 1964 and failure of the San Fernando Dams in 1971. During this period a large volume of literature has become available on seismic response and stability. Earth retaining structures, retaining walls, cofferdams, bulkheads and reinforced earth have attracted very little attention of the profession and a long search for realistic solutions is due. In the sections to follow soil deposits shall be treated first to be followed by embankments and dams, slopes and banks. And in the end, earth retaining structures shall be discussed.

3.2 Soil Deposits

In order to make an analysis of the response of any soil deposit, it is first necessary to determine (1) the surface topography, the underlying rock configuration and any irregularities in the boundaries between soil layers, (2) the types of soil comprising the deposit, (3) the characteristics of the motion by the earthquake. The response of horizontal layers to a given rock motion below has been of interest in study of liquefaction of soil deposits. In uniform deposits the shear beam method has been used. However, for layered deposits a more sophisticated method like the finite element technique is used.

In case of horizontal soil layers resting on a horizontal rock boundary, the lateral extent of the deposit has no influence on the response and the deposit may be considered as a series of semi-infinite layers (Idriss and Seed, 1968). The ground motions induced by a seismic excitation on the base are only the result of shear deformations in the soil. The resulting surface motion is relatively uniform. In case the soil modulus varies with depth in a regular manner the above solutions can be extended. Lumped mass solutions of Idriss and Seed (1970) considered variation of damping with depth.

The soil deposit of Niigata was analysed based upon the above principles by dividing the deposit into 20 layers each 3 m thick. The use of more layers did not lead to any significant changes in the computed response (Seed and Idriss, 1967).

Dezfulian and Seed (1970) studied the response of horizontal soil surfaces overlying a sloping rock profile and concluded that the maximum ground acceleration may be computed by using the semi-infinite layer analysis.

3.3 Embankments and Dams

The concept of inertia force has found favour with the engineer to such an extent that most of the problems of dynamic loading have been reduced to those of pseudo-static loading with inertia force (mass multiplied by acceleration) introduced as one of the additional forces. In a stability analysis the inertia force is introduced in such a direction that the factor of safety is reduced (Indian Standards, 1893-1975). A factor of safety of one is accepted for the loading conditions specified. There is no rational basis for adopting this procedure because the earthquakes-induced oscillatory motion in the dam and the one-directional equivalent "static force" may result in a factor of safety which may have little physical meaning.

It is recognized now (1977) that it is the displacement and stresses in the embankment rather than the numerical value of factor of safety which governs the stability of the section. Model tests have also been carried out to decipher the behaviour of typical sections. Field behaviour of dams during earthquake has also been studied. It is not possible to refer to all the literature which has appeared on the subject in the past decade. Efforts have been made to make the review as complete as possible so that a sequential story builds up.

The first study of the seismic response of earth dams appears in the published literature by Mononobe, Takata and Matamura (1936). An isoceles triangle made up of isotropic, homogeneous and elastic material on a rigid semi-infinite base was considered. As the base width of the dam is much greater than its height, the deformation due to shear is considerably larger than that due to bending. The shear stress and deformation on a horizontal section are considered uniform and the three-dimensional system was reduced to a one-dimensional shear beam. The fundamental period, T_o, is given by

$$T_o = 2.62\, H \sqrt{\rho/G} \qquad (3.1)$$

Krishna (1962) has shown that for such a model, the acceleration is maximum near the crest and it reduces in a characteristic manner towards the base. This analysis suffers from the following shortcomings: (1) the shear stresses and deformations on a horizontal section may not be uniform; (2) the side slopes do not influence the seismic response; (3) zoned sections with different material properties in different parts of the embankment are not accounted for.

Ishizaki and Hatakeyama (1962) studied two-dimensional problems by finite difference methods and found that the shear beam type approach is inadequate for predicting stresses away from the central part of the cross-section. Since the stability of the embankment is significantly influenced by the stress condition near the dam faces, the shear wedge analysis is not satisfactory.

Newmark (1965) suggested for the first time an approach to compute displacements in embankments, suitable for rigid-plastic materials like free draining cohesionless soils involving the determination of yield acceleration α g, at which sliding will begin to occur. The displacements are computed when this acceleration is exceeded. For soils in which pore pressure changes develop as a result of the shear strains, determination of appropriate value of the yield acceleration becomes extremely difficult. An alternative approach has been suggested by Seed (1966) based on (1) determination of the stresses acting on soil elements within an embankment both before and during an earthquake, (2) subjecting typical soil samples in the laboratory to the same sequence of stress changes experienced by corresponding elements in the field and observing the resulting deformations, and (3) estimating the deformations of the slope from the observed deformation of the soil elements comprising it. The method thus gives consideration to the time history of forces developed in the embankments or slopes during an earthquake, the behaviour of the soil under simulated earthquake loading conditions, and the desirability of evaluating the embankment deformations rather than a factor of safety only. Seed recommended a strain of about 13%, constituting failure in laboratory tests, for determination of properties on static and dynamic loading. This value is substantiated by Seed et al. (1975) by analysis of the upper San Fernando Dam which failed during the earthquake of Feb. 9, 1971. However, Lee and Waters (1973) investigated stability along a 5 cm wide and 3.8 m deep longitudinal crack developed along the crest on Dry Canyon Dam in an earthquake of 1952. Analysis of stability with different assumed axial strains corresponding to failure suggested that 5% strain may be an appropriate failure criterion.

Clough and Chopra (1966) applied the finite element analysis to solve the seismic response of a dam section, and Chopra (1967) compared the seismic response of a symmetrical triangular cross-section 300 ft (90 m) high with slopes of 3:1 and 5:1 and made of homogeneous isotropic and elastic material by the shear beam and finite element methods. The ground motion was N-S component of the El-Centro May 18, 1940 earthquake. Natural frequencies, mode shapes and earthquake response were compared. It was found that the first anti-symmetrical mode (in both cases) resembled a pure shear distortion. There are significant vertical displacements involved in all other anti-symmetrical modes and symmetrical modes have no resemblance to the shear type modes. Contours of shear stress due to earthquake motion only indicate the deviations from constant shear stress across the width, the basic assumption of the shear wedge analysis. The time history of dynamic stresses at the centre line and face at corresponding levels is quite different, and the seismic coefficient from the shear beam approach is larger on the average about 10 per cent to 20 per cent.

Finn and Khanna (1966) further analysed the 1 on 1.5 slope used by Chopra (1967) and the first five natural frequencies of the dam were determined by both methods. The fundamental frequency was essentially the same by either method (as were the mode shapes) indicating that the vibration on the first mode was primarily in shear. However, in higher modes the bending and extensional motions ignored by the shear slide method affected the natural frequencies and mode shapes leading to lower and more closely spaced frequencies.

To study the effect of foundation on the response of this typical section, a 108 m deep foundation with modulus of foundation different than that of the dam was considered. The natural period of the embankment resting on an elastic foundation was in general increased. Chopra and Perumalswamy (1969) obtained similar conclusions, and also found that absolute maximum stresses may increase by as much as 200 per cent in case vertical acceleration was included in the analysis. The static stresses in the sloping core dam indicated tension in the top one sixth of the core. It was concluded that central core dams were much safer. This is contrary to evidence from model tests (Clough and Pirtz 1958, Krishna et al. 1966).

Ambraseys and Sarma (1967) studied the response of homogeneous sections to the N-S component of El-Centro May 1940 ground motion and concluded that in strong motion the upper part of an earth dam near the crest is most vulnerable. This is similar to the findings of Seed (1967) who has presented an excellent review of the past practices treating stability of embankments during earthquakes. General practice in the analysis involves the computation of the minimum factor of safety against sliding when a static horizontal force is included in the analysis. The Indian Standards code of practice (1893-1975) permits a factor of safety of unity under earthquake loading conditions. The design coefficient may be selected by any one of the methods suggested by Seed and Martin (1966). Seed (1967) showed that the values of the seismic coefficients increased with increasing elevation of the potential sliding mass within the body of the embankment. The seismic coefficients varied with the height of the embankment, material characteristics, and the nature of earthquake ground motions. This type of information provided the necessary basis for analysis of deformations and for the planning of laboratory test procedures.

Seed et al. (1975) reported slides in the San Fernando Dams which developed during the 6.6 Richter magnitude earthquake of Feb. 9, 1971. In the upper San Fernando Dam (24 m high) a pseudo-static analysis of seismic stability of the embankment gave a factor of safety of 2 to 2.05 and for the lower dam, a factor of safety of 1.05. Dynamic stability analysis of the lower dam by Seed's (1966) method gave a factor of safety of about 0.8. Analysis of shear strain induced by the earthquake in the upper dam indicated an average shear strain potential of about 12 % to 16 % indicating relative horizontal downstream movements of 1.4 m to 1.8 m which was in excellent accord with the observed movement.

Dibaj and Penzien (1969a) considered the response of an earth dam to a travelling seismic wave in which case the wave length was long compared to the width of the dam. The shear stress contour lines were nearly vertical over the central region of the dam when using the travelling wave input, while they were nearly horizontal when using the uniform base input. The critical shear stress regions were near the face of the dam for the travelling wave input while they were near the center of the dam for the case of a uniform input. Uniform base motion excited only the anti-symmetric modes of a symmetrical dam. However, in the case of a travelling wave base motion, anti-symmetry no longer existed.

3.4 Model Tests and Field Response of Dams

Model tests have been conducted to study the behaviour of embankments and dams under earthquake loading. Clough and Pirtz (1958) reported the first systematic model tests for a 90 m earth and rockfill dam. The model height was 2 ft (0.6 m) with a scale ratio (λ) of 1/150. From similitude considerations for the same ratio of forces due to dead weight, water load, inertia force, and forces associated with elastic deformation and failure, it has been shown that acceleration in the model and in the prototype were equal. Also the ratio of cohesions and moduli of shear deformation in the model to the prototype was λ if the unit weight of the material in the model and in the prototype were the same. On the basis of tests on two models on a vibration table, one with central core and the other with sloping core, it was found that the latter was somewhat more earthquake-resistant than the central core type because its structure was more closely bound together. In general, the models suffered no significant changes in section up to a horizontal acceleration of 0.4 g. When the table motion was increased to more than 1 g, the model suffered only minor changes of shape. Seed and Clough (1963) reported model tests on sloping core dams 0.65 m high under empty and full reservior conditions. In a typical model, the crest settlement was approximately 2.9 % of the height of the dam with peak earthquake accelerations of 0.68 g, and 1 % of the height of the model with peak earthquake acceleration of 0.52 g.

In model studies on a shake table 5 m long x 2.8 m wide for the Ram Ganga Saddle dam 60 m high (Krishna and Prakash, 1966), the problem was to decide on the location of the core from seismic considerations.

Models with no core, central core, and inclined cores were tested. The reservoir conditions were both dry and full. The inclined core dam was found to behave better than the central core dam. A very important conclusion from this as well as from a previous study (Krishna and Prakash, 1965) was that the damage patterns in the model with core and that of the Ohno dam damaged in Kanto earthquake of 1923 (Japan Society of Civil Engineers, 1960) were identical. Typical longitudinal crack developed along the crest. This observation was substantiated by field data (Seed et al., 1975; Lee and Waters, 1973) This showed conclusively that the model tests would give an insight into the behaviour of prototype dams particularly in relation to inelastic deformations.

Another model study of a rockfill dam was performed at Roorkee (Prakash et al., 1972) for Pandoh dam in Punjab 61 m high. The scale ratio (λ) was 1/100 and the model height was 0.61 m. Elastic response of the models to a modified Koyna earthquake was studied and it was found that even within the elastic range, the test conditions in the model were adequately severe. The inelastic response was studied by comparing the damage potential of the table motion with the ground motion expected at site. The deformation of the dam profile was recorded with a special profile-meter. It was found that displacements occurred mainly at the crest. The conventional analysis showed a factor of safety of less than unity for the top quarter of the slope. The displacements obtained showed the section to be safe. The presence of a berm at a typical level affected the damage pattern in the section in a characteristic manner.

Noda, Tsuchida and Kurata (1974) tested six models with the maximum table acceleration of 200-300 gals. The maximum acceleration at the top amounted to about 1600 gals in sand models and 2700 gals in clay models. Crest settlements of 4 % to 15 % of the height of the models were observed with no sliding surface. Okamoto (1975) reported tests on 1.4 m high models subjected to sinusoidal vibrations. A berm 50 cm wide was introduced either at 70 cm or 90 cm height. It was found that for equal slopes, there must be greater acceleration for crumbling in case there was a berm and a slope with no berm experienced crumbling throughout.

On the basis of tests on six models Watanabe (1977) reported that zoning did not affect the response appreciably and the amplitude of response acceleration for the condition of full reservoir water was reduced to about 66 per cent of that for no reservoir water. Arya et al. (1977) reported model tests on a 99 m high rockfill dam tested with a scale ratio (λ) of 1/150 on the shake table available at Roorkee. Two models, one with a central core and the other with inclined core sections showed that slumping was more in an inclined core section although the tendency for separation of the shell from the core was greater with a central core. Also for the inclined core, analytical as well as experimental values of displacements agreed fairly well, if the variation of shear modulus proportional to the square root of overburden pressure was included in the analysis.

Several observations of response of earth dams subjected to earthquakes have also been collected and are summarized below.

Minami (1969) reported field response of Makio and Togo Dams which were subjected to maximum ground acceleration of 33 gals and 52 gals, respectively. It was found that ratios of maximum acceleration at the crest of the dam to the maximum acceleration of the ground increased for small earthquakes.

Atrakhova (1973) reported measurements on a 30 m high earth dam and found that the maximum amplitude of vibrations, at the top of the dam increased 1.5 to 5 times the amplitude at the base. Also the ratio of a maximum amplitude of displacement, velocity and acceleration at the top of the dam to those of the base decreased under stronger earthquakes. This observation is similar to that of Minami (1969).

Takahashi et al. (1977) tested actual dams 64.5 m to 102.8 m high. Two tests on a 95 m high dam showed that the amplitude grew remarkably near the crest of the dam, as the reservoir level rose the natural frequencies were slightly reduced, and the equivalent viscous damping factor was about 5 to 6 per cent. This dam has been subjected to a maximum crest acceleration of 100 gals in the earthquake of Sept. 9, 1969 (magnitude 7). The mean value of strain between the crest and 25 m below the dam crest was 1.5×10^{-4}. A comparison with the test results showed that the dam behaved linearly during the earthquake.

The case records of failure of the San Fernando dams have conclusively shown the myth of a factor of safety computed on the basis of pseudo-static method. A realistic analysis is therefore one which gives insight into the displacements of typical points and the settlements of the crest and the foundation. Newmark's (1965) approach is applicable to non-plastic soils in which the stress-strain relations may be assumed rigid-plastic. Also an estimate of yield acceleration is made. Seed's (1966) approach incorporates realistic properties of soils from static and dynamic loading.

Most of the subsequent analyses incorporate study of stresses, seismic coefficients, and deformations obtained by using the finite element method. The soil properties had been assumed linear and tension zones have been delineated in the section. The effect of sequential construction and nonlinear properties have not been evaluated to any extent.

It would appear that without a large computer, the analysis of an earth and rockfill dam is not possible. Also, some generalized charts for response similar to the slope stability charts of Taylor, have not been made. It is difficult at this stage to determine if such design aids can be prepared at all. It is believed that there is a need to develop an engineering approach to predict displacements of earth dams with limited use of computers and the finite element method. It may sound unfashionable or unscientific, but the need is definitely there. Therefore, published information on static stresses and on response of typical dam sections under recorded earthquakes needs to be condensed into an easily usable form. Also information on soil properties under a combination of static and dynamic loading needs to be digested into an easily usable form. Then "displacement charts" similar to "stability charts" may be developed to predict displacements. For nonlinear analyses (Dibaj and Penzien, 1969; Sharma, 1976) and sequential construction (Sharma, 1976) only a beginning has been made.

Model studies show promise. Such studies are expensive and need to be interpreted carefully. It is satisfying to see that in 1977 more such studies have been reported.

3.5 Response of Banks

Banks may be analysed by the finite element method (Idriss, 1967). The actual elastic continuums replaced by a series of elements interconnected at a finite number of nodal points. The horizontal boundaries are theoretically of infinite extent, but for purpose of analysis the soil is bounded by a rigid base and by rigid vertical finite boundaries. The input data for any computation consists of geometric description of the bank, material properties, damping ratio, and earthquake base motion. The response values (nodal displacements, velocities and accelerations, element stresses and nodal stresses, throughout the bank and the seismic coefficients of specified wedges of the bank) for the duration of the earthquake motion constitute the output of these programmes. An independent check on the accuracy of the solution is provided by the fact that the part of the bank sufficiently far from the slope and the artificial finite boundaries would then behave essentially as a semi-infinite layer (Idriss and Seed, 1967).

For a typical clay bank and for the base motion as the first 15 second record of El Centro earthquake of May 18, 1940, an examination of the values of maximum horizontal shear stress induced along four horizontal planes, it was seen that the shear stresses increased with depth in the bank. Values of the maximum seismic coefficients decreased with increasing extent of the wedge behind the face of the slope. The seismic coefficients also decreased with increasing depth of the base of the wedge below the top of the embankment. Also from the time history of stresses and seismic coefficients, the stability of the bank could be evaluated. The largest major slope slide, the Turnagain slide which occurred during the Alaskan Earthquake, and which covered a length about 2 miles (3.2 km) and a width of 600 ft to 1200 ft (180-360 m) was analysed by Seed and Wilson (1967) based upon these procedures. The effect of vertical components of motion, material properties and the geometry of the bank on the response have also been discussed (Idriss and Seed, 1967).

Kovacs, Seed and Idriss (1971), tested three banks of clay, 15 cm high undergoing horizontal accelerations of 0.05 g. It was found that a distance of six times the thickness of the adjacent layer from the crest or the toe, the test section behaved as semi-infinite layers. A comparision of the observed response with the one computed using results of laboratory measurements of soil modulus and damping characteristics provided results in good agreement with observed values.

Idriss, Seed and Dezfulian (1969) studied eight deposits with different configurations subjected to a known horizontal excitation. In two deposits, one with sloping base rock boundary and a level surface and in another with level rock boundary but sloping bank, the maximum acceleration values were almost identical throughout. A comparison of response of banks with slopes of respectively 1:1, 2:1 and 4:1 shows that the response values behind the crest of the bank are almost unaffected due to change in slope angle. Flatter slopes seem to lessen the abrupt changes in the maximum horizontal accelerations along the surface of the slope. Response accelerations increase with increase in the Young's modulus of the lower layer. Also, for a given value of Young's modulus for the lower layer, the elastic property of bank material does not affect the response acceleration appreciably.

Arango and Seed (1974) reported tests on three clay banks 15 cm high with an initial factor of safety of 1.2 and 1.4. It was observed that it was possible to induce major displacements in these banks, representing failure. A modified version of the method proposed by Seed (1966) is recommended for seismic stability analysis of saturated slopes.

Finn and Miller (1975) considered response of an infinite slope 15 m thick on a rock boundary 2 H : 1 V subjected to base accelerations of the first five seconds of the N-S and vertical components of El Centro earthquake of May 18, 1940. The response by nonlinear analysis is quite different than with linear analysis. As was shown by Finn and Byrne (1969) the location of the yielding zones depended on the strength distribution within the soil profile.

3.6 Earth Retaining Structures

In design of earth retaining structures the magnitude of earth pressures and their distribution with height must be known. Earth pressures depend on properties of the backfill material, the deformation of the structure and the consequent strains in the material retained, and flexibility of the structure. During seismic disturbance, a rigid retaining wall vibrates along with some backfill. The movements away from the backfill can take place easily whereas the resistance to its movement towards the backfill is considerably larger. Therefore, after a number of cycles of motion, the wall moves out and assumes a different position (Prakash, 1971) Such observations have been made following past earthquakes, Chile 1960, Alaska 1964 and Niigata 1964 (Seed and Whitman, 1970). Thus displacement of a wall away from the fill is a very important factor, but in almost all design procedures to date (1977) only earth pressures are computed by the same method. In the published literature, earth retaining structures other than retaining walls have not been studied under seismic conditions. Therefore, the discussion is concerned mainly with retaining walls. The question has been studied in two parts, one computation of earth pressure and two the determination of point of application of the dynamic component which equals the total earth pressure minus the static earth pressure.

The Indian Standards 1893 - 1975 incorporates a graphical procedure based upon Mononobe (1929) and Okabe (1929) formulations and recommends that the dynamic earth pressure acts at 0.55 H above the base. Matsuo and Ohara (1960) computed the lateral earth pressure in vibration considering the backfill as a two-dimensional body, and the soil to be homogeneous and isotropic. Ishii, Arai and Tsuchida (1960) developed a theory for determining dynamic earth pressures on retaining or quay walls almost similar to that of Matsuo and Ohara assuming the soil to be visco-elastic in case of fixed walls and elastic for a moving wall. Arya and Gupta (1966) theoretically obtained nonlinear lateral earth pressure distribution by assuming that horizontal accelerations vary linearly from the base to the top of the retaining wall. Prakash and Saran (1966) developed non-dimensional plots for determining the dynamic earth pressures on retaining walls holding $c - \phi$ soils assuming a plane rupture surface below the zone of tension cracks.

Madhav and Rao (1969) presented design curves for determining earth pressure coefficients as functions of cohesion, angle of internal friction, seismic coefficient, wall friction and inclination of wall and the backfill, on the basis of a pseudo-static analysis. The direction of the resultant inertia force was so optimized that the resulting pressures were maximum. Prakash and Basavana (1969) highlighted the fundamental deficiency, of Coulomb's theory and hence in Mononobe's formula that the moments of force on the rupture wedge about any point do not balance. Considering extreme cases of failure taking place along the face of the wall and on a plane surface in the backfill, it was established that the distribution of earth pressure was similar to the distribution of soil reaction on the rupture surface determined by separately maximizing the moments and pressures on the wall and proposed coefficients for determining the point of application.

Seed and Whitman (1970) have recommended an empirical formula for determining the dynamic earth pressure coefficients K_{ae}

$$K_{ae} = K_a + 3/4\ \alpha_h \qquad (3.2)$$

where K_a is active earth pressure coefficient under static case and α_h is a horizontal seismic coefficient. Scott (1973) proposed a model as a one-dimensional shear beam attached to the wall by springs. Expressions were derived for pressures, forces, and moments which might act on the wall during an earthquake. The analysis can be modified to account for the flexibility of the wall and variation of soil properties with depth. It was concluded that the pressures and moments were significantly higher than those calculated by Mononobe-Okabe method. The point of application of the earth pressures were in general around 2/3 times the hight of the wall above its base.

Aggour and Brown (1973) used a finite element model for a wall backfill system to determine the dynamic pressures on walls due to a sinusoidal ground motion. It was found that the pressure was smaller than that of a rigid wall. Nandkumaran and Joshi (1975) determined the point of application of dynamic earthpressure assuming the same rupture surface as developed under static conditions and no tension on the surface. The point of application of the dynamic increment was dependent on the geometery and the design seismic coefficient, α_h. Saran and Prakash (1977) suggested a modification to Mononobe-Okabe formula considering the mobilized angle of internal friction and angle of wall friction, both for passive and active pressures. The pressure distribution was also determined.

Several experimental studies have also been performed. Experimental studies of Matsuo and Ohara (1960) showed that the maximum earth pressure decreased with increased magnitude of the wall displacement. Ishii, Arai and Tsuchida (1960) applied sinusoidal dynamic loads on fixed and movable walls on the shake table and concluded that the distribution of dynamic earth pressure was parabolic and maximum pressure was equal to or smaller than given by Mononobe-Okabe theory. Ichihara and Matsuzawa (1973) observed during tests in a large size vibrating bin that the point of application of resultant earth pressure was 0.45 above the base for an acceleration of 0.3 g and the pressure distribution was not hydrostatic. Using a centrifuge device in laboratory tests and from field tests on 3 m high retaining walls, Aliev et al. (1973) found that the measured earth pressure had a better correlation with velocity than with acceleration.

Nandkumaran (1973) tested a 1.0 m high flexible wall and a 1 m high rigid wall under impact, and a 2.0 m high rigid wall subjected to both impact and steady state vibrations. It was found that the dynamic increment in earth pressure correlated with the peak ground velocity rather than with accelerations. The coefficients of dynamic increment K_{ae} may be represented as

$$K_{ae} = 0.016\ V \qquad (3.3)$$

where V = peak velocity (cm/sec). The point of application of the dynamic increment in case of flexible walls was about mid-height. The point of application of the dynamic increment in the case of rigid walls was at 0.4 H above the base. A mathematical model to analyse displacement of gravity type walls subjected to translation was proposed.

There is some evidence to show that a retaining wall 21 m high subjected to a ground acceleration of 0.67 g in the Koyna earthquake of Dec. 11, 1967 would fail, but when studied on the basis of peak ground velocity as proposed by Nandkumaran it would not fail.

Richardson and Lee (1975) tested small size reinforced earth walls under horizontal sinusoidal loading. It was found that the walls responded to the input motion like a non-linear damped elastic system and that the Mononobe-Okabe method gave reasonable predictions of the failure plane in the backfill material.

Information regarding dynamic passive pressure is quite limited. Sabzaveri and Ghahramami (1974) presented an incremental approach to the dynamic analysis of the passive earth pressure problem in which an initially rough retaining wall was considered to move with different values of acceleration into a dry loose sand medium. Wall rotation about the top and bottom, and wall translations were considered. It was observed that for some values of wall acceleration, the translating wall experienced the largest dynamic pressure increase. Prasad (1974) reported tests on a 30 cm high wall backfilled with sand. The dynamic K_p values were nearly 10 % smaller than the static values. Prakash et al. (1973, 1975) reported dynamic tests on rigid walls 1 m high in boulder deposits at two sites, one in India, the other in Bhutan. The dynamic passive pressures were inferred from which to determine the change in the angle of internal friction of boulder deposits.

The available information indicates that the pseudo-static method is used for computing the dynamic increment. The concept of peak ground velocity (Nandkumaran,1973) is promising. Field evidence needs to be examined to verify its applicability.

The important factors such as deformation of the wall and consequent strain pattern are not considered. The wall deformation has a significant influence on the magnitude and distribution of dynamic earth pressures. A wall tends to undergo certain displacement in an earthquake. Assuming a rigid-plastic behaviour of the soil-wall system it can be said that if the earthquake accelerations are smaller than the "yield" accelerations, no displacement of the wall would result. On the other hand if the acceleration exceeds the yield acceleration, displacements occur depending upon the time for which these conditions prevail. The present design procedures do not take into consideration the damage potential of the ground motion, the reversal of the forces, and the resulting displacements. Therefore, a need exists to assess the earthquake-induced displacements for retaining walls for more realistic design, as in the case of an embankment (Prakash, 1971). Dynamic response of other retaining structures such as coffer dams and bulkheads need attention of research workers.

3.7 Acknowledgements

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4. DYNAMICS OF BASES AND FOUNDATIONS (by D.D. Barkan and V.A. Ilyichev)

4.1 Introduction

One of the features of the modern industrial development is an increase of the importance of accompanying dynamic factors. This is due to two reasons, firstly, to the increase of capacity and number of industrial installations being intensive sources of vibration and, secondly, the development of precision devices and equipment sensitive to vibrations. That explains a considerable interest in dynamics in many branches of Engineering including Construction Engineering and, especially, Foundation Engineering Problems of Foundation Dynamics are of great importance for engineering practice.

The modern industrial installations and power plants, when generating waves, cause "a polution" of natural medium characterized by the formation in soils of vibrations being sometimes troublesome and even dangerous. The precise devices and equipement are often located in the industrial sites having powerful sources of vibration. Thus, the decrease of intensity of vibration level of the ground surface, brought about by industrial and other installations, presents the main problem in the dynamics of foundation.

One of the means for solving the problem is the use of vibroisolation of foundations, dynamic dampers and so on. However the usage of these means is not always possible from technical points of view and is not always justified economically. Therefore, it is particularly important to develop the methods of dynamic calculations as well as design of foundations ensuring the level of vibrations within allowable limits. This report is mainly devoted to this problem and the matter is presented considering the state-of-the-art of research work carried out during last ten years. In this connection much attention is given to the dynamic calculations of foundations being the sources of vibration.

4.2 On Foundation Vibrations Problem

In engineering designs the massive machine foundations are considered as rigid bodies resting on an elastic support. On the whole such a system possesses six degrees of freedom. However, as a rule the systems with a single and two degrees of freedom are considered in practice. In particular the vertical oscillation $w(t)$ is described by equation (4.1) which can be easily solved

$$m\,\ddot{w}(t) + r(t) = p(t) \qquad (4.1)$$

in which m = mass of foundation and machine; $r(t)$ = foundation base reaction; and $p(t)$ = load applied to the foundation.

The problem lies in the determination of the foundation base response. For this purpose it is necessary to represent the foundation base by some model. In soil mechanics there were proposed a lot of foundation base models for which the analysis is given, for example, in a book by Gorbunov-Posadov and Malikova (1973). In the majority of cases these models are linear, i.e., there is a linear relationship between an applied force and displacement. In the dynamics of foundations the two models are widely used, viz.:
(1) Winkler-Voigt model, i.e., set of parallel independent elastic springs and dampers.
(2) Homogeneous isotropic elastic half-space.

The linear law was confirmed by the tests more than once. For example, this is illustrated in Fig. 4.1. However, the nonlinear character of the foundation base is often discovered and this case is discussed in Sec. 4.6.

4.3 Winkler-Voigt Model

According to this model the subgrade reaction is expressed as

$$r(t) = k\,w(t) + b\,\dot{w}(t) \qquad (4.2)$$

in which k = coefficient of rigidity; and b = damping coefficient.

The main advantage of the model is its simplicity and, as a result, the possibility of obtaining closed-form solutions readily. In particular, on its basis in the USSR (1971)

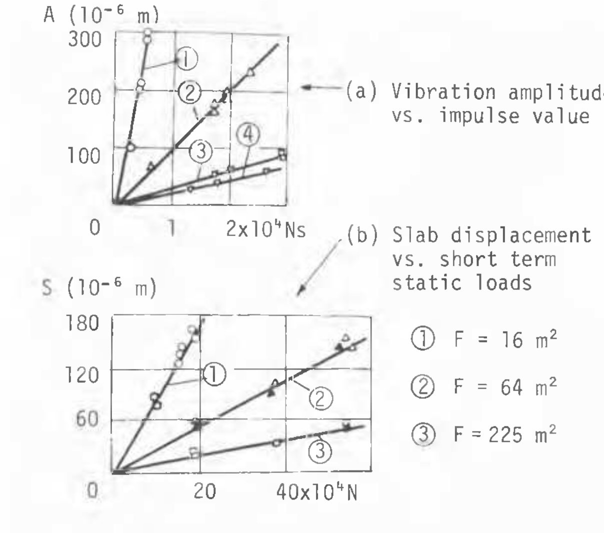


Fig. 4.1 After Lyatkher et al (1972)

and India (1966) technical codes for design of machine foundations with dynamic loads have been worked out. The Winkler-Voigt model has been widely used in monographs by Burdugan published in Rumania (1968) and France (1972) and in a book by Krashnikov (1970).

The main disadvantage of the Winkler-Voigt model is that it gives no information of the coefficients of Eq. (4.2) and, in particular, of the influence of the physical and mechanical soil properties and the dimensions of a foundation on their values. Therefore, their determination may be carried out only on the basis of special tests or by using more complicated models of a foundation base.

The model does not take into consideration the inertia mass of a soil and wave interaction of the foundation-soil system. Thus, the mass of an oscillating system is assumed to be equal to the joint mass of machine and foundation and that contradicts the test data. The latter show that the magnitude of an oscillating mass defined by tests carried out with foundations vibrated vertically is greater than m and equal to βm. Table 4.1 illustrates the values of coefficient β calculated on the basis of the experimental data.

Comparing the oscillating mass magnitude βm with the value of mass m in Eq.(4.1) it follows that at a steady state vibration the subgrade reaction (Eq.(4.2)) is to have one more item, besides the two stated ones, expressed as

$$- m_s \omega^2 W e^{i\omega t} \qquad (4.3)$$

in which $m_s = m(\beta - 1)$; and W = complex amplitude of forced vibrations of foundation.

The mass m_s is often called "an added soil mass" oscillating in phase with a foundation. In order to represent clearly this magnitude, in Table 4.1 for each test a thickness of soil layer h_s is noted for which the mass is equal to m_s. It follows from Table 4.1 that it is impossible to neglect the item of Eq.(4.3).

Apparently "the added soil mass" depends on the sizes of foundation, its embedment, geological formation of a construction site and soil properties and does not depend on the foundation mass. To all appearances, for horizontal-rotary vibrations the coefficient β differs slightly from unity. Also, in studies devoted to non-steady-state vibrations of foundations under the impact loads the researchers, as a rule, assume β = 1 when processing test data in accordance with Winkler-Voigt model.

The experimental investigations with actual and model foundations carried out for the last few years and earlier permit a definition, even if rough of damping and rigidity coefficients for foundation bases for various machines and ground conditions. The additional symbols entered into the following tables are as follows:

m_o = dropping mass; $Q_o h$ = work of dropping mass; $C_z = k/F$ = coefficient of elastic uniform compression; $\xi = b(2\sqrt{\beta mk})^{-1}$: relative damping coefficient; and β = 1 in Tables 4.2, 4.3 and 4.5.

From results of early published studies for hammer foundations ξ = 0.4 to 0.5, which differs slightly from results obtained for foundations of drop hammers for breaking scrap.

Table 4.1

Foundations tested		Soils	β	Equiv. layer thickness h_s/L	Reference
Base area F=LxL m^2	Mass m ton				
1	2.2	Loam of medium strength	1.6	0.78	Novak (1957)
7.7	13.9	Loams	1.6	2.2	Fry (1963)
4.0	8	Fine sands	1.5	0.3	Bibanov et al. (1964)
16 64 225 225	31 370 1300 2090	Saturated sands	3.7 2.7 4.1 2.2	0.65 0.61 0.61 0.67	Lyatkher et al. (1972)
1.0 1.44 1.96 3.24 4.84	8.5 8.7 9.1 10.1 11.2	Loess & loessial sandy clay, sand	1.6 1.7 1.8 1.9 2.0	3.0 2.08 1.53 0.93 0.62	Ilyichev and Taranov (1976)
2	10	Strong clays	4.2	6.7	Barkan & Shaevitch (1976)

Note: All foundations rested on surface except the last one which was embedded 1.2 m.

Table 4.2 Dynamic Characteristics of Foundation Base of Drop Hammers for Breaking Scrap (after Kharkiv Research Institute PROMSTROYPROJEKT)

Soil Conditions	Number of foundations tested	$Q_o h$ MNm	F m^2	m ton	ξ	C_z MN/m^3
Fine sands (natural water cont., avg. density)	3	4.0	158	2650	0.47	35
	3	1.8	50	660	0.53	42
Loessial loam (humid, avg. density)	2	1.8	50	660	0.46	33
Saturated sandy clays and loams	3	2.9	105	2300	0.47	40

Note: Foundations were embedded to depths of 5 m to 7 m.

Table 4.3 Dynamic Characteristics of Foundation Base for Vibro-Isolated Hammer Foundations (after Barkan, 1969)

Soil conditions	m_o ton	F m^2	m ton	C_z MN/m^3
Fine-grained sands (humid, avg. density, permissible soil pressure of 200 kN/m^2)	2	51	340	52
	3	64.8	600	73
	4	71.5	670	66
	5	77.4	730	63
	16	90	980	61
Sandy clay with lenses of loam and sands (natural water content, permissible soil pressure of 200 kN/m^2)	1	20.2	104	56
	2	31.4	245	95
	3	43.7	332	83
Sandy clay (avg. density, saturated, permissible soil pressure of 200 kN/m^2)	2	31.4	245	63
	5	50.4	650	91
Boulder clay of hard consistency (water table below bottom of foundation, permissible soil pressure of 400-600 kN/m^2)	1	20.2	104	224
	2	31.4	245	317
	2	31.4	245	144
	3	43.7	322	270

Note: Depths of embedment varied from 3.3 m (for 1-ton hammers) to 10 m(for 16-ton hammers).

Table 4.4 Dynamic Characteristics of Bases for Foundation Slabs Resting on Fine-Grained Saturated Sands (after Lyatkher et al.,1972)

F m^2	m ton	ξ	C_z MN/m^3
16	31.4	0.34	73
64	380	0.34	62
225	1320	0.40	50
225	2140	0.44	42

Notes: (1) Oscillation of the slabs was caused by vertical impulsive loads;
(2) resonant curve was defined by calculation.

Table 4.5 Dynamic Characteristics of Saturated Fine-Grained Sands and Those at Natural Water Content (from studies of vibrations of test foundations by Krasnikov, 1970)

Characteristics of foundations		Vibrator eccentric moment	Sand at natural water content		Saturated sand	
F m^2	m ton	Nm	ξ	C_z MN/m^3	ξ	C_z MN/m^3
(a) Due to free vertical vibrations						
2.3	3.4		0.27	47	0.26	36
4.0	8.0		0.30	43	0.24	36.5
6.6	16.4		0.32	40	0.23	36
25.6	128.0		0.30	39	0.31	31
(b) Due to forced vertical vibrations						
2.3	3.4	2.1	0.22	56	0.21	49
		4.1	0.21	50	0.22	42.5
		9.2	0.18	38.2	0.15	29.2
		15.2	0.12	28.4	0.17	26.6
4	8.0	2.1	0.30	52.2	0.27	51.0
		4.1	0.25	46.8	0.17	50.0
		9.2	0.19	43.2	0.16	41.5
		15.2	0.16	39.6	0.12	32.8
6.6	16.4	2.1	-	-	0.26	47.8
		4.1	0.19	48.4	0.27	43.0
		9.2	0.18	46.8	0.15	42.3
		15.2	0.17	48.6	0.11	37.7

From the data noted above, it follows that for saturated sands the values of ξ and C_z are less than for sands with natural water content. When the exciting force (vibrator eccentric moment) is increased, the above two values decrease under forced vibrations. The value of ξ for free vibrations is greater than for forced vibrations.

Values of ξ and C_z obtained from measurements of vibrations of machine foundations in operation may be used directly for design of new foundations on similar soils. The values of ξ and C_z for other soils can be obtained by using the correlations between elastic and strength characteristics. The studies of these correlations have been the subject of much attention for the last few years. For example, Barkan, Trofimenkov and Golubtsova (1974) have established a correlation between C_z and permissible soil pressure as well as between modulus of elasticity and modulus of deformation. There is relationship between transverse wave velocity and standard penetration resistance, defined by Imai and Yoshimura (1971).

The accumulation of data for the above correlation will allow, without special investigations of elastic properties, the determination of elastic properties from strength characteristics, which are evaluated by standard methods on all construction sites.

4.4 Homogenous Isotropic Elastic Half-Space Model

The notable work by Reissner (1936) has stimulated the development of a new branch in the dynamic theory of foundations. Now the range of solved problems concerning vibrations of rigid plates on the half-space is extensive. The harmonic and impulsive loading as well as vertical, horizontal, rotary and rocking vibrations have been considered. Foundation shape may be varied and is not restricted to circular. The review of the above-mentioned problems can be found in the work by Borodachev (1969), and also in the works by Richart, Hall, Woods (1970), Seimov (1975), Lyatkher, Yakovlev (1976). The character of massive body behaviour described below using as an example the vertical vibrations are just and true for a majority of similar problems. Naturally, the numerical characteristics differ but there is no essential change in the foundation behaviour.

The vertical harmonic footing oscillation is defined by Eq.(4.1) which after omitting factor $\exp(i\omega t)$ is expressed as

$$- m\omega^2 W + R = P \qquad (4.4)$$

in which P = amplitude of exciting force; W and R = complex amplitude of foundation oscillation and reaction force, respectively.

There is a relation between W and R given by

$$W = R\, f_o^* \qquad (4.5)$$

in which $f_o^* = f_1^*(\omega) + i f_2^*(\omega)$

The identical rearrangement of this formula suggested by Hsieh (1962) gives a relationship between R and W analogous to Eq.(4.2)

$$R = k(\omega)\, W + i b(\omega)\, W \qquad (4.6)$$

in which

$$k(\omega) = f_1^* \left(f_1^{*2} + f_2^{*2} \right)^{-1}, \quad b(\omega) = -f_2^* \left[\omega \left(f_1^{*2} + f_2^{*2} \right) \right]^{-1}$$

The amplitude of foundation vibrations can be found by equation

$$- m\omega^2 W + \left[f_1^*(\omega) + i f_2^*(\omega) \right]^{-1} W = P \qquad (4.7)$$

if f_1^* and f_2^* are known. Thus, the problem lies in the determination of these functions and, principally, all solutions of this type differ just in the manner of the construction f_1^* and f_2^*.

For the first time Shekhter (1948) gave corrected functions f_1^* and f_2^* and she drew a conclusion of the possibility of approximating the half-space model by a single-degree-of-freedom system with specially chosen characteristics. When formulated exactly the contact problem of harmonic oscillation of the massive plate was solved by Borodachev (1965).

The available results of the steady-state vertical oscillation of the rigid circular foundation of radius a resting on the homogenous isotropic elastic half-space can be given in brief in the following way.

At a relatively low-frequency oscillation, when $\omega a/C_2 < 1$ the distribution of contact stresses under the rigid plate is close to the static one. The displacement of the rigid plate defined by functions f_1^* and f_2^* and found from the exact and approximate solution of the static distribution of contact stresses are nearly idenfical (Borodachev, 1965). When frequencies of oscillation are increased the distribution of contact stresses approaches the uniform one.

By increasing frequences the functions f_1^* and f_2^* oscillating approach zero. Therefore the functions $k(\omega)$ and $b(\omega)$ defined by Eq.(4.6) do not change as monotonous when frequencies are being increased (Fig. 4.2). Hence the response curve for the massive foundation has after the main resonant peak also the additional ones (Fig. 4.3).

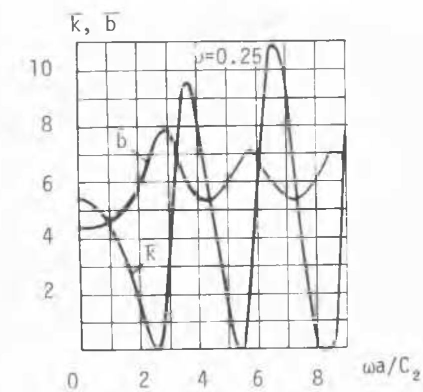


Fig. 4.2 After Ilyichev (1975)

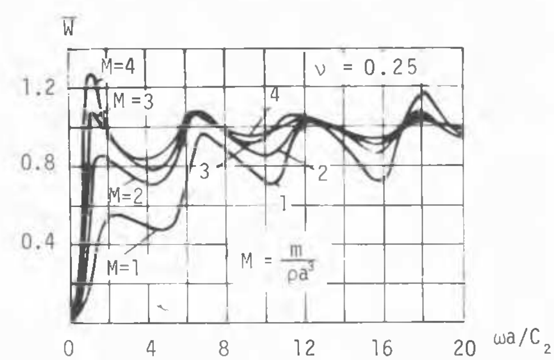


Fig. 4.3 After Shekhter (1969)

The character of the functions noted above determining the dynamic displacement of the rigid plate under harmonic load does not allow one to approximate adequately, even if roughly, these displacement functions by means of the single-degree-of-freedom system. However in the restricted but practically important range of frequences $\omega a/C_2 < 1.5$ such a comparison can be made and gives the following results.

The parameters of vertical harmonic vibrations of the massive body lying on the half-space may be found by solving the equation

$$-(m + m_s)\omega^2 W + i\omega b W + kW = P \qquad (4.8)$$

The coefficients of the above equation are determined as suggested by Lysmer (1965)

$$m_s = 0, \quad b = 3.4a^2\sqrt{\rho G}/(1-\nu), \quad k = 4Ga/(1-\nu) \qquad (4.9)$$

in which G = shear modulus; ρ = soil mass density; and ν = Poisson's ratio.

Scheiter (1967) gave, in addition to relations (4.9), a formula defining m_s

$$m_s = \rho a^3/(1-\nu) \qquad (4.10)$$

The beginning of studies of non-steady-state vertical oscillation of rigid plates dated much later and these investigations have not been so complete as steady-state problems. The blow of a rigid cylinder against the elastic half-space was considered by Guttzwiller (1962). The non-steady-state oscillation of mass of the half-space were studied by Ilyichev (1964), Lysmer and Richart (1966) and Seimov (1975).

When being subjected to a suddenly applied loading on the massive body the contact stresses are distributed uniformly at the beginning and then they tend to a distribution corresponding to a static contact problem when the massive body vibrations are damped (Seimov, 1975).

In solving the non-steady-state problems it is convenient to use a method of conjugation of two different dynamic systems as was made for steady-state vibrations. If the displacement $w_o(t)$ of the weightless rigid plate under an instantaneous pulse is known, the oscillation of a massive body is defined by solving both Eq.(4.1) and the equation

$$w(t) = \int_o^t r(t_1)\, w_o(t - t_1)\,dt_1 \qquad (4.11)$$

The diagrams of functions $w_o(t)$ when accepting some simplified assumptions are illustrated in Fig. 4.4 (Ilyichev, 1973).

The function $w_o(t)$ over the range of $0 < \nu < 0.4$ can be substituted by the exponential function and that results in the model coinciding with Eq.(4.9). At a non-steady-state oscillation "the added soil mass" in contrast to the harmonic vibration is strictly equal to zero for $\nu < 0.5$ since the function $w_o(t)$ is not equal to zero when t = 0. At the value of $\nu > 0.4$ the approximation of Eq.(4.9) is unfit.

However this contradiction may be removed by introducing a more complicated model. According to the investigations of Ilyichev the model with "one-and-a-half degrees of freedom" shown in Fig. 4.5 describes satisfactorily the displacement of the rigid plate under the action of harmonic and impulse load. The model parameters are as follows:

$$b_1 = \pi a^2 \rho C_1, \quad k_1 = 2\pi a \rho C_1^2$$

$$m_2 = \pi^2 \rho a^3/\beta_1^2 B, \quad k_2 = \pi G a/B \qquad (4.12)$$

$$b_2 = 2\pi^2 a^2 \rho C_2/\beta_1 B$$

in which C_1 and C_2 = velocity of longitudinal and transverse waves.

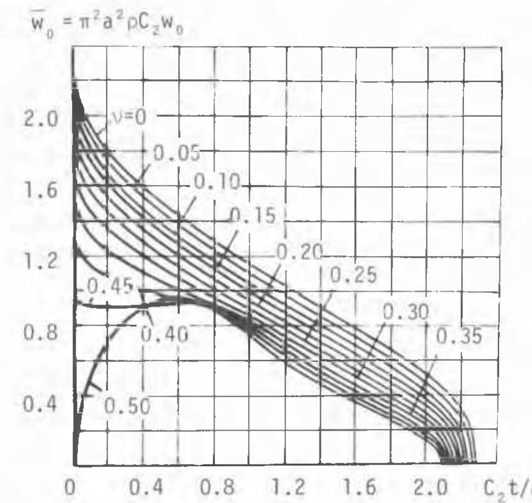
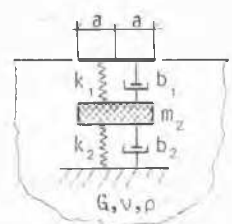


Fig. 4.4



C_2: Velocity of transverse wave

Fig. 4.5

Table 4.6

ν	β_1	B
0.00	0.60	1.68
0.25	1.79	1.32
0.50	2.22	1.23

The upper part of the system represents a contribution to the dynamic behaviour of the rigid plate created by longitudinal waves. The lower part of the system represents the contribution to the dynamic behaviour created by transverse together with Rayleigh waves. For an incompressible medium (ν = 0.5) where the longitudinal waves do not exist the model is simplified and turns into the single-degree-of-freedom system that is true both for harmonic and transient oscillation.

In-situ experimental studies aiming at the check of the forced vertical vibration theory based on the elastic half space model have been carried out by Girard and Picard (1970) in France as well as Yamamoto, Seki, Suzuki (1971) in Japan for the last few years. While processing the test results the authors come to the conclusion that the half-space model is not confirmed by tests in situ. In the main the disagreement between the theoretical predictions and test results is that there is a considerable difference in amplitude in the resonance zone. The values of test amplitudes proved to be from 2 to 5 times the ones predicted by the theory. The theoretical and experimental values of resonant frequencies have failed to agree as well. In author's opinion the reason of this disagreement lies in the fact that in reality the soil mass under the foundation is not homogenous in depth and due to the reflection of waves from the layers below, the real energy dissipation is less than the theoretical one.

4.5 Prediction of Oscillation of Actual Foundation, Soil Mass and Surrounding Structures for Real Soil Conditions

It is suggested to determine the functions f_1^* and f_2^* on the basis of in-situ tests. This idea is the basis of a new branch in experimental dynamics of foundations which can be formulated as follows. The properties of linear foundation-soil system should be studied on the basis of the experimentally discovered response of this system on the unit input either impulsive or harmonic. For the first case it is defined an impulsive transient function (ITF) and for the second-transfer function (TF). From mechanical standpoint the functions define the displacement of any chosen points (or its velocities, acceleration) at given inputs. The potentialities inherent in the formulated trends have not been entirely realized and there are only some example of its usage. Ho and Burwash (1960) constructed TF experimentally for the weightless rigid plate of about 7 in. (18 cm) in diameter using for this purpose tests with sand.

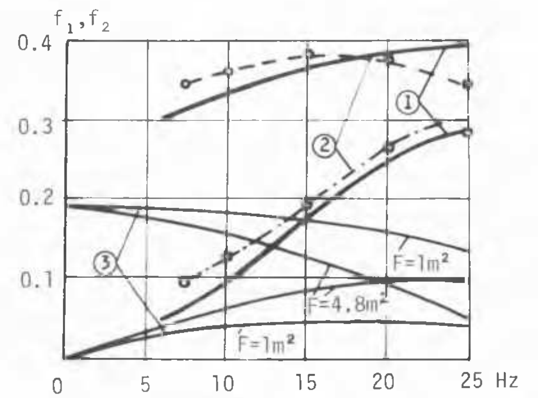
Lytkher et al (1972) realized the method of defining TF for the massive foundation by ITF obtained experimentally. Then TF was used for processing within a single degree-of-freedom system as if there were an experimental amplitude-frequency characteristic of the massive foundation. Thereby one test has been substituted for the other. Ilyichev and Taranov (1976) have constructed TF for the rigid plate on the basis of in-situ tests with foundations having a base area of 1 to 4.84 m². In the course of the test the amplitude of exciting force P, amplitude of foundation oscillation W_o and phase displacement γ between P and W_o, over the range of frequencies of $5Hz<\omega/2\pi<25Hz$ have been measured. The following formulas were used for calculations of the transfer functions:

$$|f_o^*(\omega)| = W_o\left[(m\omega^2 W_o)^2 + 2m\omega^2 W_o P\cos\gamma + P^2\right]^{-1/2}$$

$$\alpha = \tan^{-1}\frac{P\sin\gamma}{m\omega^2 W_o + P\cos\gamma} \qquad (4.13)$$

$$f_1^*(\omega) = |f_o^*(\omega)|\cos\alpha,\quad f_2^*(\omega) = |f_o^*(\omega)|\sin\alpha$$

The results are shown in Fig. 4.6.



① Experiment, ② prediction for F = 4.4 m², ③ half-space theory for ν=0.25

Fig. 4.6

Using the results of the vibration tests with a small-scale foundation Ilyichev and Taranov (1976) have developed an experimental method for predicting the level of harmonic vibration of the large-scale foundation, soil and surrounding structures as well as for determining the distribution of contact stress under a foundation. This procedure is as follows:

The contact area of the soil-foundation system is divided into N cells each having the area of F_k, k = 1, 2,... N. The integral equation corresponding to the dynamic contact problem of a rigid plate under given force $1\cdot\exp(i\omega t)$ can be solved by collocation approach which leads to the system of algebraic equations

$$\begin{aligned} &\sum_{k=1}^{N} X_k\delta_{jk} = f_o^*(\omega) \\ &\sum_{k=1}^{N} X_1 = 1,\quad j = 1, 2,\dots, N \end{aligned} \qquad (4.14)$$

Thus we determine X_k and $f_o^*(\omega)$. The coefficients δ_{jk} define the amplitude and oscillation phase in the central point of the cell j due to the harmonic uniform loading applied to the cell k. These coefficients are experimentally obtained on the construction site.

Then, Eqs. (4.7) and (4.5) are used for determining the vibration amplitude of the foundation and the reaction force R under its base. The contact stresses are determined by the formula

$$\sigma_k = R\, X_k / F_k \tag{4.15}$$

If there is a need of knowing the vibration level at any point ℓ of a soil or structure the coefficients $\delta_{\ell k}$ should be found out experimentally. Having the values X_k the displacement of this point ℓ is

$$W_\ell = R \sum_{k=1}^{N} X_k \delta_{\ell k} \tag{4.16}$$

Two examples illustrating the above method are shown in Figs. 4.6 and 4.7.

4.6 Influence of Nonlinear Base Characteristics

From the above it follows that linear theories of foundation vibrations wherein the soil is represented either by the Winkler-Voigt system or half-space model do not agree with test data. Apparently, one of the reasons of this lies in a disregard of nonlinear foundation base behaviour already discovered by the first tests. By now it is not given much attention to their study. The main works on this subject have been carried out by Novak (1957), Funston and Hall (1967).

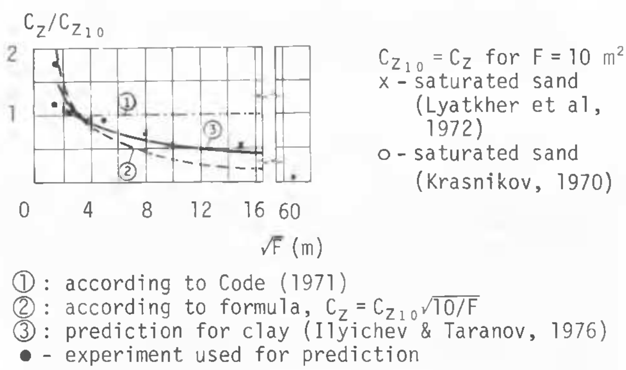


Fig. 4.7

The test by Barkan and Shaevitch (1976) have shown that soils are characterized by specific nonlinearity. It was stated that the elastic and dissipative term of foundation base reaction at vertical forced vibrations generated by the vibrator with a given moment of unbalanced masses was approximately linear over the whole range of frequences and amplitudes. However, the proportional relationship between the foundation base reactions and amplitudes of motion or velocities depends on the moment of the unbalanced masses of the vibrator. When the latter increases the coefficients decreases respectively. Thus, the nonlinearity of foundation base is defined by the relationship between the reaction and moment of unbalanced masses $m_o\varepsilon$ or limiting amplitude, $A_\infty = m_o\varepsilon/\beta m$, corresponding to the inifinite value of frequency.

So, it may be expected that the moment of unbalanced masses being constant the response curves of forced vibrations are defined by linear theory. In fact this has been confirmed on the basis of 17 response curves recorded by different researchers on various test foundation in diverse soil conditions. In all the cases the response curves calculated with account of relationship between base reaction and limiting amplitude coincided well with the test results.

Fig. 4.8 (Barkan and Shaevich, 1976) shows the relationship between resonant frequency

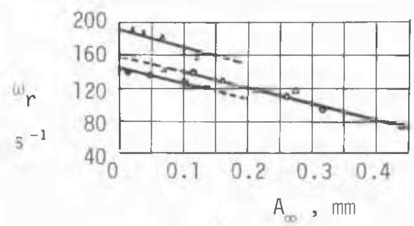


Fig. 4.8

and limiting amplitude for different tests. It may be seen that there is a linear relation as follows:

$$\omega_r = \omega_r^0 - a_r A_\infty \tag{4.17}$$

in which ω_r = resonant frequency or roughly, natural circular frequency of foundation without regard for the influence of nonlinearity, i.e., at the values of A_∞ being very small, for example, less than 6 μ. The relationship of Eq.(4.17) is true for the values of A_∞ being less than 0.5 mm to 0.7 mm typical for machine foundations. Coefficient a_r proved to be equal to 190 mm^{-1}sec^{-1} and the same for all the experiments. The tests show that the relationship between damping coefficient and limiting amplitude differs from the linear one as well. However, very few test data are available to generalize this relationship.

4.7 Conclusions

At the present time not all the problems, wherein engineers are interested in practice, are solved satisfactorily. In particular, it is not quite clear how to assign elastic and damping characteristics to pile foundations.

The problem of quantitative evaluation of the influence of foundation embedment on its dynamic characteristics remains unsettled. Now there is still no trustworthy method for definition of stiffness and damping coefficients which stipulate the separate estimation of the influence of the base area of foundation and lateral foundation surfaces on values of the coefficients and there is not quite enough experimental data.

The practical usage of more elaborate soil models in comparison with an elastic half-space is a matter of great mathematical difficulties. Things being as they are the best way out of the situation may be the usage of numerical methods for solution of the dynamic problems of a solid medium. At this it may be possible to take into consideration the foundation embedment, soil layering, its non-elastic properties, and so on. At present there is a considerable information already available in such notable works on the subject as those by Seed et al(1975), Hall and Kissenpfennig (1975) and Lysmer et al (1975).

From an experimental point of view the horizontal rotary oscillation of foundations is not studied enough.

At the Fifth World Conference on Earthquake Engineering in Rome (1975) the above mentioned problems were discussed in regard to the associated problem of structure-soil interaction under seismic effects.

In conclusion it should be noted that at present the dynamics of foundations is being developed rapidly. The necessity for predicting more exactly and completely the dynamic behaviour of machine foundations and surrounding soil always brings forward new problems to be resolved by specialists in the field of dynamics. However, it appears that researchers have to revise conventional concepts and revert constantly to the true kernel of the dynamic foundation problem, i.e., the evaluation of the response of the foundation base under the dynamic effect.

5. SOIL-STRUCTURE INTERACTION
(by F.E. Richart, Jr.)

5.1 Introduction

The dynamic response of bases and foundations resting directly upon soil masses was treated in Chapter 4. The motions of the foundation depended upon the magnitude, direction, and frequency of the applied loads, the geometry of the foundation-soil contact system, and on the dynamic properties of the supporting soil.

The response of structures supported by foundations on soils may also be influenced by the dynamic motions at the foundation-soil interface. A study of this problem begins with evaluation of the dynamic response of the foundation, then determining how this "flexible support" influences the dynamic behavior of the structure. For example, for tall buildings supported on soft soils and subjected to earthquake excitation, the foundation motion is generally different from the free-field motion and may include an important rocking component in addition to a horizontal component of motion. Thus the term "soil-structure interaction" describes the effects contributed by foundation-soil motions which cause the response of structures to differ from the response calculated on the basis of foundation motion equal to the free field motion. The difference in structural response depends on the characteristics of the free-field ground motion, the properties of the foundation system, and the properties of the structure.

5.2 Response of Foundation-Soil Systems

The dynamic behavior of a particular foundation can be evaluated most accurately by field tests on the prototype. However this is seldom possible because of the expense and construction schedules. A modification of this procedure involves full-scale dynamic tests of buildings, from which the foundation motions may be measured and their contributions evaluated with respect to the motions of the structure. Field tests on model foundations at the site can provide useful basic information, which must then be extrapolated to represent the prototype condition. Any extrapolation must be based on a sound theoretical basis. For extrapolations, and for first estimates of foundation response, solutions based on representing soil as an elastic medium have been found useful.

5.2.1 Foundations at the Surface of an Elastic Half-Space

The basic concepts and developments of this type of theoretical procedure were covered in Chapter 4. However, it is sometimes convenient to have the approximate, frequency independent, spring and damping coefficients available. These quantities are introduced into Eq. (5.1) for evaluation of the one-degree-of-freedom dynamic response of a mass-spring-dashpot system from

$$m\ddot{z} + c\dot{z} + kz = Q(t) \qquad (5.1)$$

In Table 5.1 (see Richart, Hall, and Woods, 1970), r_o is the radius of the circular foundation, m is the mass of the foundation, and G, ν, and ρ are the shear modulus, Poisson's ratio, and mass density (=unit weight/g), of the elastic half-space.

Table 5.1 Spring and Damping Coefficients for Rigid Circular Foundation Resting on Elastic Half-Space

Mode of Vibration	Spring Coeff. k	Damping Coeff. c	Mass Ratio B	Damping Ratio $D=\frac{c}{2\sqrt{km}}$
Vertical (z)	$\frac{4Gr_o}{1-\nu}$	$\frac{3.4r_o^2}{1-\nu}\sqrt{\rho G}$	$\frac{1-\nu}{4}\frac{m}{\rho r_o^3}$	$\frac{0.425}{\sqrt{B_z}}$
Sliding (x)	$\frac{8}{2-\nu}Gr_o$	$\frac{4.6}{2-\nu}r_o^2\sqrt{\rho G}$	$\frac{2-\nu}{8}\frac{m}{\rho r_o^3}$	$\frac{0.288}{\sqrt{B_x}}$
Rocking (ψ)	$\frac{8Gr_o^3}{3(1-\nu)}$	$\frac{0.8r_o^4\sqrt{\rho G}}{(1-\nu)(1+B_\psi)}$	$\frac{3(1-\nu)}{8}\frac{I_\psi}{\rho r_o^5}$	$\frac{0.15}{(1+B_\psi)\sqrt{B_\psi}}$
Torsional (θ)	$\frac{16Gr_o^3}{3}$	$\frac{4\sqrt{B_\theta \rho G}}{1+2B_\theta}$	$\frac{I_\theta}{\rho r_o^5}$	$\frac{0.50}{1+2B_\theta}$

In order to use the above expressions for the case of rigid rectangular foundations of width, b, and length, L, an equivalent circular radius can be calculated from

$$r_o = \left[\frac{bL}{\pi}\right]^{\frac{1}{2}}$$ for Translation (Vertical or Sliding),

$$r_o = \left[\frac{bL^3}{3\pi}\right]^{\frac{1}{4}}$$ for Rocking about a horizontal axis at mid-length of the base, and

$$r_o = \left[\frac{bL(b^2 + L^2)}{6\pi}\right]^{\frac{1}{4}}$$ for Torsional Motion about a vertical axis through the centroid of the base.

5.2.2 Theoretical Solutions for Embedded Foundations

Cylindrical foundations of radius, r_o, and embedded a depth, d, into an elastic material have been evaluated by elastic theory (Tajimi, 1969), by the finite element method (Lysmer and Kuhlemeyer, 1969), and by an approximate method (Novak and Beredugo, 1972; Novak, 1974) wherein a series of thin elastic sheets were substituted for the material along the vertical sides of the foundation. In each case it was assumed that perfect adhesion was developed between the foundation and elastic material.

Embedded Foundations Supported by an Elastic Half-Space:

A part of the procedure developed by Novak (1974) is presented here, even though it is approximate, because of the flexibility in accounting for different values of shear modulus for the backfill material, G_s, and for the base material, G. The important parameters involved in the solution are illustrated in Fig. 5.1. Vertical and torsional modes of vibration can occur as uncoupled modes, thus they may be considered separately. For vertical vibrations the spring and damping coefficients are,

$$k_{zz} = G\, r_o \left(C_z + \frac{G_s}{G}\frac{d}{r_o} S_z\right) \text{, and}$$

$$c_{zz} = r_o^2 \sqrt{\rho G}\left(\overline{C}_z + \frac{d}{r_o}\, \overline{S}_z \sqrt{\rho_s G_s / \rho G}\right) \qquad (5.2)$$

In Eq. (5.2), $C_z \approx 4/(1-\nu)$, $\overline{C}_z \approx 3.4/(1-\nu)$, $S_z = 2.7$, and $\overline{S}_z = 6.7$, and d is the depth of embedment. The spring and damping coefficients for torsional vibration have a similar form whereas the expressions for coupled rocking and sliding motions are more complex.

If the foundation is firmly embedded into a uniform soil, $G_s = G = \text{const.}$, and if $\nu = 0.40$, the spring constant from Eq. (5.2) becomes

$$k_{zz} = G\, r_o \left(6.7 + 2.7\frac{d}{r_o}\right) = k_z\left(1 + 0.4\frac{d}{r_o}\right) \qquad (5.3)$$

in which k_z is the value taken from Table 5.1. This type of equation indicates that

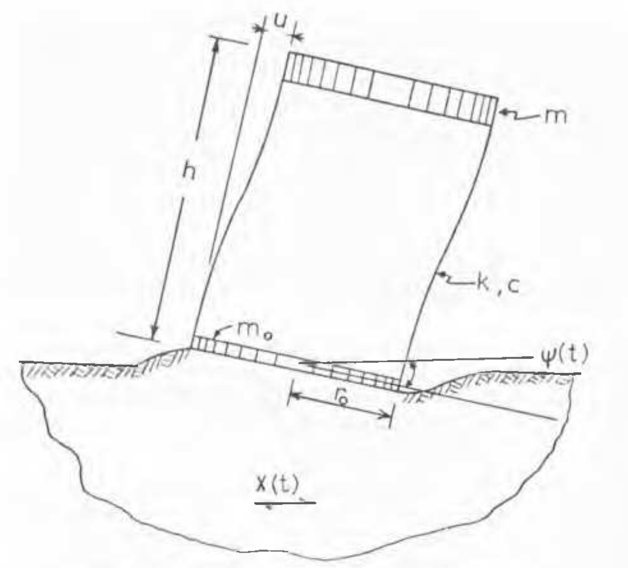


Fig. 5.1 Structure-Soil System

the resonant frequency will be increased as the embedment ratio, d/r_o, increases, and from the increased damping associated with increasing d/r_o it is evident that the amplitude of motion at resonance will be reduced. These theoretical indications are consistent with experimental results obtained from tests of model foundations (Novak and Beredugo, 1972; Erden and Stokoe, 1975; Petrovski, 1975). Novak's (1974) solution may also be applied to evaluate the effect of embedment on the coupled rocking and sliding motion of a rigid foundation. However, this type of vibration is strongly influenced by the mass ratios (B-values in Table 5.1) of the foundation for rocking and sliding motions.

The lateral restraint developed by motion of the vertical face of embedded foundations against the soil depends upon the rigidity, G_s, of the soil, and upon *maintaining* the soil-foundation contact throughout the useful life of the vibrating system. Stokoe and Richart (1974) found the behavior of a foundation embedded into a clayey silt soil to behave as if it rested on the surface, whereas a foundation embedded in sand responded as an embedded foundation. Apparently higher temperatures in the foundation block plus vibration history caused a crack to remain open at the vertical soil-foundation contact surface for the foundation embedded in the clayey silt material.

Embedded Foundations Supported on an Elastic Layer:

Real soils consist of layers which usually increase in stiffness with depth. Thus the conditions of an elastic half-space usually do not exist in natural situations. A better approximation to real soils is obtained by considering the material beneath a foundation to be represented by a series of elastic horizontal layers of constant thickness.

The first approximation to the real situation consists of a foundation supported by a single elastic layer of thickness H resting upon a rigid subbase. This problem was treated by Warburton (1957) for circular foundations supported at the surface of the single elastic layer, and his solution was adopted by Novak and Beredugo (1972) for evaluation of the stiffness, C_z, and damping, $\bar{C}_z$, coefficients to be used with Eq. (5.2). As the elastic layer becomes thinner, the spring constant *increases* approximately as $C_{z\ell} = C_z(1 + r_o/H)$. However, the geometrical damping decreases because the elastic waves radiating from the foundation are reflected back from the rigid layer. Thus the damping coefficient, $\bar{C}_{z\ell}$, for the foundation on the elastic layer is *reduced* from that for the foundation on the elastic half-space, C_z, as shown for the same foundation mass in Table 5.2. Also shown in Table 5.2 is the percent critical damping, $D_{z\ell}$, which is a function of both $\bar{C}_{z\ell}$ and $C_{z\ell}$.

Table 5.2 Spring and Damping Coefficients for Foundation Resting on an Elastic Layer over a Rigid Base

H/r_o	∞	4	3	2	1
$\bar{C}_{z\ell}/\bar{C}_z$	1.00	0.35	0.19	0.11	0.066
$D_{z\ell}/D_z$	1.00	0.31	0.16	0.09	0.044

For small values of H/r_o, a resonant condition may be developed at a frequency of $f = v_p/4H$, in which v_p is the compression wave velocity in the elastic material. Waas (1972) showed the importance of material damping in suppressing this layer resonance effect.

The dynamic stiffness of foundations on a single elastic layer is also increased for the sliding (x) and rocking (ψ) vibrations. Kausel and Roesset (1975) have indicated that the spring constants are increased as

$$C_{x\ell} \simeq C_x\left(1 + \frac{1}{2}\frac{r_o}{H}\right), \text{ and}$$

$$C_{\psi\ell} \simeq C_\psi\left(1 + \frac{1}{6}\frac{r_o}{H}\right) \tag{5.4}$$

where C_x and C_ψ are as given in Table 5.1. Because these increases in spring constants are not as strongly influenced by the r_o/H factor as was the vertical case, it would follow that the damping terms would decrease smaller amounts than does $\bar{C}_{z\ell}$ as the ratio r_o/H decreases.

5.3 Effect of Foundation Flexibility on Response of Structures

Standard methods of structural analysis permit evaluation of the first mode natural period, T_o, of a building considered to be *fixed* at the foundation level. If the foundation rotates or translates during the time of building oscillation, the first mode natural period, T, of the coupled system will be greater than T_o. The increase depends upon the parameters involved in the dynamic behavior of the structure and of the foundation system. This problem has been treated by numerous investigators, but the following discussion will conform with the approach given by Veletsos (1976). The Veletsos paper also contains a list of recent references on this topic.

A simple structure-soil system is shown in Figure 5.1. This single-story structure has the mass, m, of the structure concentrated at a single floor level at a distance of h above the contact zone between the rigid circular foundation mat (mass, m_o) and the soil. The weightless inextensible columns deflect in shear to provide an elastic stiffness, k. Supporting the system is a semi-infinite elastic body representing the soil, which is described by its shear modulus, G, Poisson's ratio, ν, and mass density ρ. The structural system is excited by horizontal earthquake motions applied at the base.

Dimensionless parameters may be developed from the quantities in Fig. 5.1 which describe the structural system. Several of these parameters are described briefly, in decreasing order of importance. The wave parameter, σ, is a measure of the relative stiffness of the supporting soil and the structure. It is expressed as

$$\sigma = \frac{v_s T_o}{h} \tag{5.5}$$

in which v_s is the shear wave velocity in the soil. The ratio h/r_o is a significant parameter in relating the rocking and translated modes of foundation vibration and structural vibration. The frequency parameter, $f_o t_1$, is the product of the fixed base natural frequency, $f_o = 1/T_o$, of the structure and t_1, a characteristic time of the excitation. Damping is introduced as D_f the percent of critical damping for the foundation system including both geometrical and material damping, and D_s is the percent critical damping for the structure in its fixed-base condition. The relative mass density for the structure and supporting medium is described by

$$B_s = \frac{m}{\rho\pi r_o^2 h} \tag{5.6}$$

In some cases the ratio m_o/m of mass of foundation to mass of the structure can be important.

From studies of influence of soil-structure interaction on the natural period for building vibrations, Jennings and Bielak (1973), and Veletsos and Meek (1974) found this natural period to be given approximately by the equation

$$T = T_o\left[1 + \frac{k}{k_x}\left(1 + \frac{k_x h^2}{k_\psi}\right)\right]^{\frac{1}{2}} \tag{5.7}$$

The values of k_x and k_ψ in Eq. (5.7) are theoretically frequency-dependent, but for a study treating the soil as a half-space, Veletsos (1976) found little error introduced if k_x and k_ψ were taken as the expressions given in Table 5.1. For purposes of showing the results graphically as shown in Fig. 5.2, Veletsos (1976) used the parameter $\phi\sqrt{B_s}$, in which

$$\phi = \frac{1}{\sigma}(h/r_o)^{\frac{1}{4}} \qquad (5.8)$$

The effective damping ratio of the structure-soil system, D_t, is given approximately by

$$D_t = D_f + \frac{D_s}{(T/T_o)^3} \qquad (5.9)$$

in which damping of the foundation, D_f, includes both the geometrical, D, (from Table 5.1) and material, D_m, damping. D_s represents the damping ratio of the structure. For viscoelastic materials the material damping may be expressed in a variety of forms (see Richart, Hall, and Woods, 1970). Some equivalent terms are (for small values of damping)

$$D_m = \frac{\delta}{2\pi} = \frac{\tan \delta_L}{2} \qquad (5.10)$$

in which δ is the logarithmic decrement, and δ_L is the loss angle. Because the geometrical demping for rocking (D_ψ from Table 5.1) of a rigid shallow foundation can be very small for tall, slender buildings, it is necessary to consider the influence of material damping on response of the total system. Figure 5.3 shows the variation of the foundation damping factor, D_f, as functions of the h/r_o ratios, and for material damping values of $\tan\delta_L = 0$ and 0.10 ($D_m = 0$ and 0.05). A special case with $B_s = 0.15$ and $D_s = 0$ was considered. However, it is important to note the large increase in foundation damping contributed by material damping, particularly for the higher h/r_o values.

5.4 Conclusions

This brief discussion of soil-structure interaction has emphasized the importance of the foundation motions upon changes in the natural frequencies and damping of structure-soil systems. For buildings with a large h/r_o ratio, the rocking motion of the foundation and material damping in the soil are important factors.

Table 5.1 includes expressions relating to the dynamic behavior of rigid circular foundations on the elastic half-space. This is a limiting condition seldom realized in practice, but is a useful reference for evaluation of the influence of real conditions. A brief discussion of the embedment of foundations noted that embedment should increase the resonant frequency and decrease the amplitude of dynamic foundation motions. The influence of a finite thickness of the elastic soil layer between the foundation and a

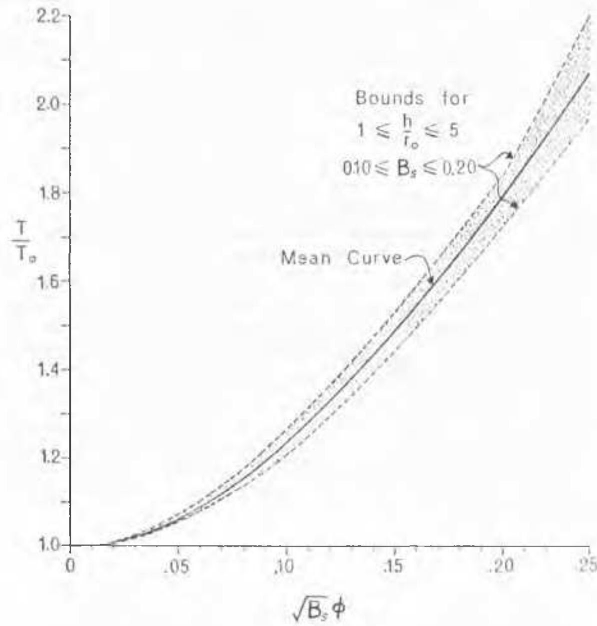


Fig. 5.2 Effective Natural Period, T, of Structure-Soil Systems ($\tan \delta_L = 0$, $D_s = 0$)

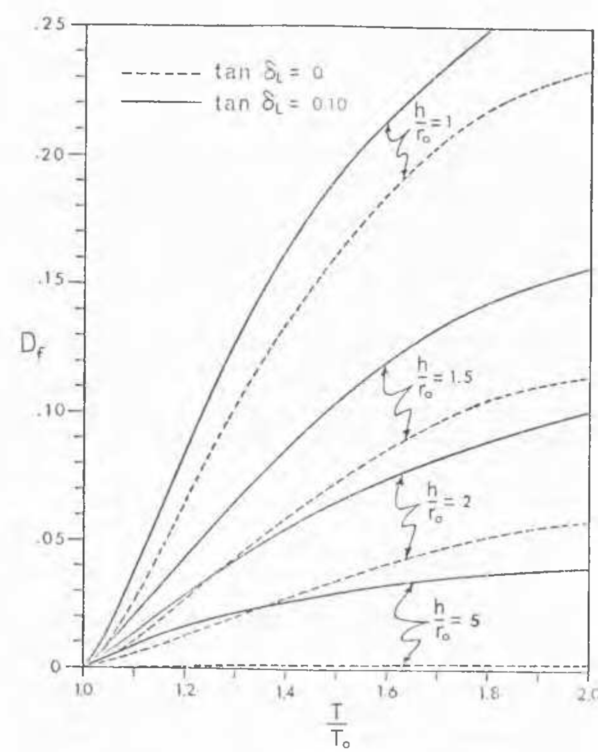


Fig. 5.3 Foundation Damping Factor, D_f, for Structures Supported on Viscoelastic Half-Space ($\tan \delta_L = 0.20$, $B_s = 0.15$, $D_s = 0$) (from Veletsos, 1976)

rigid rock base is to increase the resonant frequency, and because geometrical damping is reduced, the amplitude of vibration at resonance may be radically increased. For real soils which are usually layered and often have G-values increasing with depth, it should be expected that amplitudes of motion at resonance would be greater than those predicted from the half-space theory.

Finally, for estimating the influence of soil-structure interaction by use of Eq. (5.7) it should be noted that k_x and k_ψ must be evaluated to correspond with real conditions, which may involve foundation embedment, layering of the underlying soil, nonlinear soil reaction, or an effective resistance to rocking motion developed by a flexible foundation. For special cases of unusual significance finite element solutions may be developed to include these special conditions (see Seed, Lysmer, and Hwang, 1975).

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Note: J.SMFD, and J.GED, Proc. ASCE are, respectively, Journal of the Soil Mechanics and Foundation Division, and Journal of the Geotechnical Engineering Division, Proceedings of the American Society of Civil Engineers. WCEE is the World Conference on Earthquake Engineering.