Session 4

Soil Dynamics and Its Application to Foundation Engineering

Dynamique du Sol et son Application aux Travaux de Fondations

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INTRODUCTION

The forty-four papers from twenty countries cover a wide variety of subjects which may be classified as follows:

(1) Dynamic stress-strain relationships for soils
(2) Liquefaction of soils
(3) Seismic response of soil deposits and embankments
(4) Foundation vibration
(5) Dynamic compaction
(6) Application of impact and vibration to construction and in situ tests
(7) Environmental problems involving ground vibration
(8) Similitude requirements for dynamic model tests

DYNAMIC STRESS-STRAIN RELATIONSHIPS FOR SOILS

On the basis of results of completely reversed undrained cyclic shear tests on saturated sands and clays, Shibata, Sato and Soelarno (4/40)* present simple expressions for the shear moduli and damping ratios. For sands, a linear relationship between $\sqrt{\sigma_c}/G$ and $\gamma/\sqrt{\sigma_c}$, in which G is the shear modulus, γ is the shear strain and σ_c is the consolidation pressure, is shown to hold regardless of the consolidation pressure (0.25 to 0.90 kg/cm^2) and the type of loading conditions: constant axial stress, constant radial stress and constant mean principal stress. [The right side of Eq. (1) should read $G_o\tau_f/(\tau_f + G_o\gamma)$.]

Hara and Kiyota (4/13) describe a new Kjellman-type simple shear apparatus for dynamic tests. The movable pedestal on which a disk-shaped specimen is placed consists of two parts having approximately equal surface areas. In order to minimize effects of non-uniform stress distribution within the specimen due to the lack of complementary shear stresses along the periphery of the specimen, only the load applied to the inner part of the specimen is measured. It is claimed that shearing strains can be measured over a wide range from 0.001 percent to 1.0 percent so that effects of shear strain levels on the shear modulus and damping ratio can be investigated. Typical test results of sands, clays, and mudstones are reported.

Sherif, Ishibashi and Ling (4/39) report that the damping ratios of undisturbed samples of a soft marine clay of low sensitivity (Northeast Pacific red clay) are nearly independent of the number of cycles from 10 to 6500 during strain-controlled cyclic torsion tests.

Pender (4/28) shows that the response of soil to small-strain cyclic loading can be realistically described by a five-parameter model. This is achieved by adding one parameter to his four-parameter model that has been developed to describe large-strain static behavior of soil. The original four parameters, which can be determined from conventional static tests, are the stress ratio at critical state, the location of the critical state line in the stress space, the slope of the e-log p curve for swelling, and the slope of the e-log p curve for virgin compression. The new fifth parameter is the elastic shear modulus that can be determined from measurements of the in situ shear wave velocity. It is noteworthy that the proposed model enables one to predict realistically the effect of strain amplitude on the dynamic shear modulus and on the damping ratio.

In order to predict dynamic response of a horizontal deposit of saturated sand during earthquakes by means of a lumped mass analysis, Finn, Lee and Martin (4/10) present a sophisticated model involving seven empirical parameters. Because the model represents a complete cycle of nonlinear stress-strain relationship during loading, unloading and reloading, the model can simulate nonlinear, strain-dependent and hysteretic damping of sands in a realistic manner, provided that appropriate tests are conducted. Excess pore pressures during undrained shear can be predicted on the basis of the conditions of constant volume and constant total vertical stress, in such a way that the reduction in volume due to cyclic shear is canceled by rebound due to the reduction in the effective stress.

Tinoco (4/44) attempts to express excess pore

* The numbers in parentheses refer to the paper numbers in Proceedings Volume II.

water pressures during undrained loading by three parameters which can be determined from static tests in the laboratory. One of the parameters is identical to Skempton's parameter B. The two other parameters are independent of strains before failure. As the author admits, the proposed method seems unable to give satisfactory prediction of pore pressure development for cyclic shear tests of saturated sand. [The parameter D in Eqs. (10) and (11) should be multiplied by $\sqrt{2}$.]

Laboratory cyclic test results related to subgrade design are presented by Edris and Lytton (4/8) on cohesive soils and by Descornet (4/6) on granular soils. Edris and Lytton propose methods to predict resilient moduli and residual strains of compacted cohesive soils on the basis of soil suction, temperature, stress state, and number of load cycles. An important change in soil behavior is reported to occur when the water content is about two percent dry of optimum. Descornet presents results of unreversed cyclic triaxial compression tests on twelve granular soils having widely different gradations. The residual strain is expressed as a function of the confining stress, resilient strain, and a parameter called "susceptibility" which depends on the void ratio and water content.

Imai (4/15) presents extensive results of seismic investigations conducted at 244 locations in Japan. The down-hole method was employed in which the wave generating energy was provided by horizontal hammering of a wooden plank placed on the ground, dropping a weight, or blasting. The shear wave velocities, v_s, are plotted against the standard penetration test blow counts (N-values), pressuremeter test results, and the unconfined compressive strengths, q_u. Fairly good correlations appear to exist between log v_s and log N for $N > 2$, and between log v_s and log q_u for $q_u > 0.5$ kg/cm^2.

Some results of the down-hole method are also given by Hara and Kiyota (4/13), who report good agreement between the in situ shear moduli of a stratum of silty clay and the shear moduli at low strains [presumably 0.001 percent] of undisturbed samples of the soil determined with the simple shear apparatus that has been mentioned previously.

LIQUEFACTION OF SOILS

In an effort to predict the liquefaction resistance of a horizontal sand deposit on the basis of cyclic triaxial tests on isotropically consolidated specimens, various relationships have been proposed to evaluate the influence of the coefficient of earth pressure at rest on the liquefaction resistance of saturated sands as summarized by Seed, Arango and Chan (1975). In one of the relationships, the shear stress amplitude required to cause liquefaction in a given number of cycles, τ_ℓ, is proportional to the initial effective mean principal stress, as follows:

$$\tau_\ell \propto \frac{1 + 2K_o}{3} \bar{\sigma}_{vo} \qquad (1)$$

in which K_o is the coefficient of earth pressure at rest and $\bar{\sigma}_{vo}$ is the initial vertical effective stress.

Ishihara, Iwamoto, Yasuda and Takatsu (4/16) present convincing experimental evidence that Eq. (1) is valid not only for a K_o smaller than one ($K_o = 0.5$) but also for a K_o greater than one ($K_o = 1.5$). The experiments were conducted on saturated, fine, medium dense sand using a torsional shear apparatus capable of performing completely reversed cyclic shear tests on hollow cylindrical specimens with zero lateral strain. The authors derive Eq. (1) on the basis of a concept that the increase in pore water pressure during cyclic shear leading to complete liquefaction may be divided into two parts: one part due to cyclic shear and the other due to a change in the horizontal total stress. When K_o is smaller than one as in the case of a normally consolidated deposit, the horizontal total stress increases during liquefaction process thereby leaving smaller room for pore pressure buildup due to cyclic shear compared to an isotropically consolidated case. On the other hand, when K_o is greater than one as may be possible in a compacted sand, the horizontal total stress decreases during liquefaction process allowing greater amount of pore pressure development due to cyclic shear.

Shen, Vrymoed and Uyeno (4/38) describe some results of undrained cyclic tests on saturated sands with varying amounts of fines. For a given "void ratio of the sand structure, which is defined as the ratio of the volume of voids and fines to the volume of sand fraction alone, the resistance of liquefaction is claimed to increase with an increase of the fines, provided that the void ratio of the sand structure stays below the maximum void ratio of the pure sand. It should be noted, however, that for a given "void ratio of the sand structure," the dry density increases as the percentage of fines increases. More detailed information about the method of specimen preparation would be valuable.

Flores-Berrones and Dawson (4/11) describe some field evidences of liquefaciton caused by shallow, small magnitude ($M = 4.5 \sim 5.5$) earthquakes. Typical evidences of liquefaction are cracks in recent alluvial deposits within a distance of 6 km from the zone of energy release. Considering previous experiences in Japan (Kuribayashi and Tatsuoka, 1975; Nasu, Kotoda and Wakamatsu, 1976) and elsewhere (Youd, 1977), magnitude 4.5 is probably the lower limit for liquefaction to occur.

Finn, Lee and Martin (4/10) describe a method of nonlinear effective stress analysis to predict the development and dissipation of pore water pressures in a horizontal deposit of saturated sand during earthquakes. The response of a typical soil profile of Niigata, Japan, to irregular ground motions is computed by this method. Parametric studies show that the drainage conditions or soil permeabilities may have significant effects on the liquefaction potential and the time required to cause liquefaction.

Ferrito and Forrest (4/9) describe a Monte Carlo technique for predicting the probability of liquefaction on the basis of available data on soil strength and earthquake ground motion. The method enables one to obtain graphs which show the factor of safety and probability of liquefaction as a function of earthquake magnitude, distance from the fault, and relative density of the soil at the site. The authors correctly state that both the damage from liquefaction and the probability of occurrence must be reviewed together in order to evaluate the risk of liquefaction at a site.

Nishiyama, Yahagi, Nakagawa and Wada (4/24) describe a systematic method to predict liquefaction potential of level ground during earthquakes. The method consists of two steps, and is similar to the procedure proposed by Seed and Idriss (1971).

Ivanov (4/17) discusses liquefaction of saturated cohesionless soils subjected to dynamic loads from the viewpoint of consolidation. Compressible pore water containing occluded gas may be treated, and a concept of "vibrocreep" process is proposed. The author also describes a method of explosive sounding to predict liquefaction potential, and presents data showing average vertical strains due to explosion (settlement divided by thickness) plotted against relative density.

There are three papers dealing with undrained cyclic strength of saturated clays. Sherif, Ishibashi and Ling (4/39) describe results of strain-controlled torsional shear tests on undisturbed samples of a soft marine clay of low sensitivity (Northeast Pacific red clay). The ratio of the undrained strength after cyclic straining to the initial strength decreased with the number of cycles and the cyclic strain amplitude, as expected, but appeared to be nearly independent of the frequency of cyclic straining over a range between 0.025 Hz and 10 Hz.

Ogawa, Shibayama and Yamaguchi (4/26) describe stress-controlled cyclic triaxial tests on a remolded silty clay [at a frequency of 2 Hz]. The test results are in essential agreement with those reported previously as described in Section 2.7 in the State-of-the-Art Report. More information about the method of pore pressure measurement would be useful.

Soils supporting offshore gravity structures are subjected to series of cyclic loads during storms with quiet intervals during which dissipation of excess pore pressures may take place. Such loading and drainage sequence was simulated in the laboratory by Brown, Andersen and McElvaney (4/4), using completely reversed cyclic simple shear tests and unreversed triaxial shear tests. A series of cyclic loading followed by drainage tends to increase the undrained shear strength of normally consolidated samples, but has little effect on overconsolidated samples.

Centrifugal tests on models of offshore gravity platforms subjected to simulated wave loading are described by Le Tirant, Luong, Habib and Gary (4/19) and Rowe, Craig and Procter (4/34). The test results show that there is a danger of wide-spread liquefaction of sand below the structure although the zone of liquefaction may be limited under more favorable conditions.

SEISMIC RESPONSE OF SOIL DEPOSITS AND EMBANKMENTS

Seismic shear stresses developed in typical soil deposits in Japan are estimated by Shibata, Sato and Soelarno (4/40) with a nonlinear harmonic wave analysis, and by Nishiyama, Yahagi, Nakagawa and Wada (4/24) with a wave propagation method using shear moduli and damping ratios which depend on both strain amplitude and confining stress. The shear stress reduction factor, r_d, which is the ratio of the shear stress amplitude in horizontal ground to that in fictitious, perfectly rigid ground, is shown to decrease with depth at considerably greater rate than the values proposed by Seed and Idriss (1971). The lower values of r_d by Nishiyama et al are probably due to the fact that most of the soil profiles used by the authors contain soft alluvial clays of considerable thicknesses.

As mentioned in Section 2.5.1 of the State-of-the-Art Report, investigations of earthquake hazards at hydraulic-fill structures in seismically active areas in the U.S.A. have recently been undertaken. Marcuson, Krinitzsky and Kovanic (4/21) describe an example of such investigations concerning the Fort Peck Dam, Montana. The method, which is essentially the same as the dynamic analyses proposed by Seed, Lee, Idriss and Makdisi (1975), involves extensive field investigations, soil sampling, cyclic triaxial tests, and finite element analyses. It is concluded that no part of the dam will experience shear strains exceeding 5 percent during earthquakes considered possible at the site.

Critical of the current trend that only the vertically travelling shear wave is considered in evaluating the seismic response of soil deposits and structures, Asada (4/1) attempts to draw our attention to the importance of the surface waves. On the basis of field observations and multi-reflection analyses of ground motions during moderate earthquakes and microtremors at a certain site, the author concludes that the horizontal component of dispersive Rayleigh or Love wave is at least as important as the horizontally polarized shear (SH) wave, and that simultaneous observations of seismic ground motions at three surface locations (tripartite observation) should be recommended to recognize the surface waves.

There are two papers dealing with the seismicity and soil conditions in Lima, Peru. As a case study of seismic microzonation, Martinez and Romani (4/22) present a seismic risk potential map of Lima, Peru. The map containing five zones of varying degrees of seismic risk is said to have been derived from seven other maps showing the topography, geology, geomorphology, hydrology, soil types, seismicity, and damage during previous earthquakes. However, there is no explanation of the method to prepare the seismic risk map from the other maps, except that the method is not based on quantitative evaluation of soil rigidity and other data.

Carrillo-Gil (4/5) plots the seismic intensities recorded in Lima, Peru, during the Lima earthquake of 1974 against the Young's modulus of the soils [presumably at shallow depths]. Although it is claimed that the seismic intensity increases as the Young's modulus decreases, such relationship does not appear to exist when the data for individual soil type are treated separately.

In the second part of the paper by Ivanov and Sinitsyn (4/17), Sinitsyn presents an intriguing and highly condensed discussion of elastic or plastic seismic waves causing overturning of buildings founded on two-layer soil.

FOUNDATION VIBRATION

Sperling and Hausner (4/43) present the results of extensive measurements of dynamic motions of about 400 machine foundations resting on uniformly graded sands. Whether or not the sands vibrated causing settlement of the foundations was used as a criterion of stability. The foundations were judged stable when the velocity amplitude stayed below a certain limit which was a function of the "density index" of the sands. More information about the definition of the density index would be valuable.

Sankaran, Subrahmanyam and Sastri (4/35) present analytical solutions and model test results of machine foundations resting on the ground surface and subjected to horizontal vibration. Radiation damping, material damping, and Coulomb friction are considered in the analysis. The predicted natural frequencies and maximum amplitudes on the basis of "reference tests" are claimed to agree with the observed values. It should be noted, however, that the size and weight of all the foundations including those designated as the reference tests covered relatively narrow ranges, i.e., the radii and weights were from 34 cm to 51 cm and from 404 kg to 1168 kg, respectively.

Ramiah, Chickanagappa and Ramamurthy (4/31) describe an approximate method to estimate the effects of embedment of machine foundations on the vertical spring constant and damping coefficient. When compared to a series of field tests, the method overestimated the resonant frequency by about 100 percent although agreement was better concerning the resonant amplitude. As discussed in Section 5.2.2 of the State-of-the-Art Report, the effect of embedment may vary depending on the manner in which the soil makes contact with the sides of a foundation. This aspect of foundation embedment, however, is outside the scope of this paper.

Savidis and Richter (4/36) describe an analytical method to determine dynamic interference between two buildings on rigid rectangular foundations resting on elastic half space. The boundary conditions are approximately satisfied by specifying the displacements at discrete points at the base of the foundations. Numerical results are presented for several cases in which one of the two foundations is subjected to harmonic excitation.

Dungar, Eldred and Haws (4/7) present results of finite element analysis of a typical offshore platform of the gravity type. Dashpots are provided along the boundaries of the soil to absorb the radiated energy. Although the natural period of this particular system is out of the range of significant dynamic response to North Sea storm waves, the authors warn that there may be cases in which dynamic response becomes important.

Mirza (4/23) describes some laboratory test results of 30 cm x 30 cm model footings with or without piles subjected to steady-state vibration. Some formulas and charts for estimating the natural frequency of machine foundations are presented, but their reliability is not discussed on the basis of observations of full-scale foundations.

Prakash and Chandrasekaran (4/29) describe three methods for predicting the natural frequency in horizontal vibration of vertical piles with a lumped mass attached at the top. The methods are (1) a single degree of freedom idealization, (2) an equivalent cantilever idealization, and (3) a modal analysis of discrete lumped mass and spring models in which the horizontal subgrade modulus is constant or proportional to the depth below ground surface. The first mode natural frequencies predicted by the third method agreed well with those observed in the field tests, provided that the pile head was assumed free to rotate in the analysis.

Rodriguez Ortiz and Castanedo (4/33) present some results of parametric studies of dynamic response of a mass supported by a single point-bearing pile. Seismic excitation is represented by horizontal harmonic motion in bedrock. Stratified soil can be replaced by an equivalent homogeneous soil by weighting the layer thickness.

Novak (4/25) presents the results of linear analyses of dynamic response of vertical piles for which geometrical damping is accounted for. With the simpler of the two theories used, variation of soil properties with depth can be dealt with, enabling one to compare friction piles and point-bearing piles. Sway-rocking motions of pile foundations are analyzed by combining the complex stiffnesses for vertical and horizontal motions. On the basis of field experiments with small pile foundations [not reported in this paper], it is claimed that the theory can give satisfactory results if the properties of that part of the soil that participate most in the motion of the pile are used and if the motion of the pile tip is included for the vertical response.

Prange (4/30) shows that the inertia of a transducer can cause significant errors in measuring vibration if the frequency to be measured exceeds several tens of cycles per second. Two methods are proposed to compensate the inertia effects: (1) mathematical compensation by inverse inertia function or analog computer, or (2) mechanical compensation by an auxiliary vibrator.

DYNAMIC COMPACTION

Concerning vibratory compaction of soils, Selig and Yoo (4/37) recognize four possible mechanisms: (1) particle vibration, (2) impact, (3) strength reduction, and (4) cyclic straining. As far as compaction by vibratory rollers of soils other than dry or submerged granular materials are concerned, the fourth mechanism is considered the most important. Full-scale field tests, laboratory model roller tests, and theoretical analyses are conducted in order to provide a rational basis for selecting rollers and to determine the best operating conditions. It is interesting to note that a linear system of two degrees of freedom could represent the compaction characteristics of a vibrating roller, despite the fact that the static load-displacement relationship was definitely nonlinear.

There are three papers dealing with Menard's "dynamic consolidation method" which consists of dropping a heavy weight from considerable heights (up to 40 m) onto the ground in order to compact it to considerable depths in tens of meters. Hansbo (4/12) describes a case in which rock fill up to 30 m thick was treated by the method using a 40-ton or 12-ton weight. Because the pressuremeter test was not applicable except in upper part of the fill where the rock had been pulverized during compaction, the author tried seismic refraction measurements, plate loading tests, and measurements of the deceleration of the falling weight. The slopes of the dynamic pressure vs. settlement curves determined from the deceleration measurements are in the same order of magnitude as those obtained by the plate loading tests. It is concluded that the deceleration measurements can replace the expensive, static plate loading tests. It should be noted, however, that either method is limited to shallow depths, and that seismic methods using deep holes are required, as they have been done for rock fill dams in Japan, if more reliable information is sought concerning the elastic moduli in deep parts of a rock fill.

Brandl and Sadgorski (4/3) present results of field measurements of subsurface stresses and pore pressures in organic fine-grained soils caused by a 17-ton weight falling from 5-m to 16-m heights. The pressure bulbs for the peak dynamic stress were generally similar to those computed by the elastic theory for static loads, except at shallow depths where the dynamic stresses were considerably smaller than the static stresses. The "rest stress" immediately following impact was negative at a shallow depth (fissuring zone) while it was positive at a greater depth (compaction zone). But no clear line separating the two zones could be established. The attenuation characteristics of the stress with distance generally agreed with previous theoretical and experimental results.

Biarez, Fournier and Rudelle (4/2) describe laboratory test results of "dynamic consolidation" of six cohesive soils. Impact of a falling weight on a soil specimen in an oedometer caused a sudden compression and a subsequent reduction in the compression index. Laboratory vane shear test results show that the manner in which the shear strength changes after impact may be widely different for different soils.

APPLICATION OF IMPACT AND VIBRATION TO CONSTRUCTION AND IN SITU TESTS

It has largely been a trial-and-error procedure to use diesel hammers to drive piles under different conditions. Rempe and Davisson (4/32) show that the performance of diesel pile driving hammers can be predicted accurately by wave equation analysis if combustion effects are properly accounted for. The method enables one to calculate the maximum stroke for a given driving condition and select proper piles and driving equipment without costly trial-and-error process in the field. A judicious application of the proposed method is expected to contribute to rational planning and control of pile driving.

Despite its limitations the Standard Penetration Test (SPT) has recently received a renewed support as a useful index of liquefaction resistance of sands (Seed, Arango and Chan, 1975; Schmertmann, 1977). Kovacs, Evans and Griffith (4/18) describe an interesting study of the effects of some operational details of the SPT on the impact velocity of the hammer, which is used as an index of the energy delivered. The impact velocity was significantly influenced by the number of turns of a rope around the cathead (winch), the condition of the rope, the method of releasing the rope, and the fall height. The authors recommend that an impact velocity or delivered energy should be specified for the SPT, in such a way that the existing correlations involving the N-value are least disturbed. It is suggested that the standard impact velocity could readily be calibrated for each rig and operator by adjusting the fall height of the hammer.

Păunescu (4/27) describes some field experiences related to application of vibration (8 to 50 Hz) to thrusting of piles, boring, soil sampling, and installation of cast-in-place concrete piles. Vibration amplitudes and contact pressures required to thrust piles and other objects having cross-sectional areas from 80 to 900 cm^2 into medium to fine sands are given. The field test data show that vibration is more efficient than "vibro-percussion" for thrusting piles into saturated sand, but that vibro-percussion is more efficient than vibration for clay. More information about the physical and mechanical properties of the soils would be valuable.

ENVIRONMENTAL PROBLEMS INVOLVING GROUND VIBRATION

Noise and ground vibration caused by pile driving, compaction by high energy impact, and demolition of structures have become a serious environmental problem in urban areas. Although noise during pile driving can be reduced by surrounding the rig with a wall, isolation of ground vibration requires more costly subsurface barriers.

There are three papers dealing with ground vibration. Recognizing that ground vibration during pile driving is essentially due to the Rayleigh wave generated at the pile tip, Lo (4/20) describes a practical procedure to predict maximum particle velocities on the surface of a single-layer soil on the basis of rated hammer energy, cross-sectional area of the pile, coefficient of attenuation of the soil, and dynamic penetration resistance of the soil. It is noted that the pile length had little effect on the amplitude of the induced particle velocity. When applied to pile driving operations at two project sites, the procedure gave satisfactory estimates of the maximum particle velocity at varying distances from the piles being driven.

Skipp and Buckley (4/41) present interesting data of ground motions (particle velocity) for three types of energy release: (1) impact of cooling towers demolished by explosive removal of the supporting legs, (2) explosion, and (3) rigid body impact. The data show that only a small part of the kinetic energy of the demolished structures were released as source of ground vibration, whereas relatively high percentage of the energy of rigid body impact contributed to ground vibration.

Haupt (4/14) discusses the effectiveness of a buried concrete wall for isolating the Rayleigh wave on the basis of numerical calculations using the two-dimensional finite element method. The effect of a concrete wall for reducing the vertical amplitude of the Rayleigh wave is essentially governed by the vertical cross-sectional area of the wall rather than by the proportion. The wall can reduce the vertical amplitude of the wave to about 20 percent if the cross-sectional area exceeds about 1.4 times the square of the wave length.

SIMILITUDE REQUIREMENTS FOR DYNAMIC MODEL TESTS

Similitude relationships for model experiments involving dynamic loading and pore pressure dissipation are discussed by Le Tirant, Luong, Habib and Gray (4/19), Rowe, Craig and Procter (4/34), and Smits (4/42). According to Smits identical strains are said to develop in a prototype and a geometrically similar model if the effective stress paths are similar while the ratios of the principal stress difference to the mean effective principal stress is kept identical. Specifically, it is required that $(G_s - G_f)gL/(1+e)$ be equal for the prototype and model, where G_s and G_f are the specific gravity of the solid and the pore fluid, g the gravitational acceleration, L the linear dimension, and e is the void ratio.

According to Le Tirant et al and Rowe et al who use centrifugal models to simulate geostatic stresses, the similitude requirements in dynamic response and creep behavior cannot be satisfied simultaneously because the period must be proportional to the linear dimension for identical dynamic response whereas the same time scale must be used for identical creep behavior. There appears to be a reasonably good chance to satisfy most of the similitude requirements for dry or submerged granular soils subjected to cyclic loading because these soils are essentially free from creep behavior under moderate stresses and their dynamic moduli and strengths are nearly independent of loading frequency. In order to circumvent the difficulty in simulating the creep effect in cohesive soils, Rowe et al suggest modification of displacements and softening in the model on the basis of undrained cyclic tests on soil elements.

There is a consensus that the similitude in pore pressure dissipation or drainage can be satisfied by using identical consolidation time factor for prototype and model. This has been achieved by replacing pore water in the model with a viscous fluid such as molasses (Smits), oil (Rowe et al) or glycerin (Yoshimi and Tokimatsu, 1977). An alternative method is to use a soil of finer grain size for the model as mentioned by Le Tirant et al and Rowe et al.

Smits describes some results of model tests (not centrifugal) for a permeable caisson dam subjected to cyclic storm loading. Although the study appears to be in a preliminary stage, there is an encouraging indication that a critical load could be simulated by the model tests.

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