Session 10

Soil Dynamics — General Report

Dynamique des Sols

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INTRODUCTION

The four technical topics assigned to this session by the Organizing Committee were:

(1) Evaluation of dynamic soil properties

(2) Design of machine foundations

(3) Damage by vibrations and protection of buildings

(4) Effect of dynamic loads on strength and deformation properties of soils

As a result, few of the papers submitted to the session dealt explicitly with earthquakes and offshore structures -- the two engineering problems that have dominated research and publications in soil dynamics during the four years since the last International Conference. However, many of the papers dealing with Topic 4 were motivated by these.

The outline of this General Report follows the listing of assigned topics, although brief mention will also be made of other matters touched upon by some of the papers. The report concerning topics 1 and 4 has been prepared by the Co-Reporter and the remainder of the report (except part of the sub-section on Wave Propagation under "Other Topics") by the General Reporter.

Soil dynamics has been a major topic of a number of major internationlly-attended conferences since IX ICSMFE in Tokyo in 1977, including:

Dynamic Geotechnical Testing, Denver, Colorado, June 1977, ASTM Special Technical Publication 654.

Earthquake Engineering and Soil Dynamics, ASCE. Geotechnical Engineering Division Specialty Conference, Pasadena, Calif., June 1978. 3 volumes

2nd Int. Conf. Micozonation, San Francisco, Calif., Nov-Dec 1978. 3 volumes published by Univ. Washington, Seattle, Wash.

2nd U.S. National Conf. on Earthquake Engineering, Stanford University, Calif., August 1979; Earthquake Engineering Research Inst.

2nd Int. Conf. on the Behaviour of Off-Shore Structures, London, England, August 1979. 3 volumes published by BHRA Fluid Engineering, Cranfield, U.K.

Soils Under Cyclic and Transient Loading, Swansea, Wales, January 1980. Proceedings published by Univ. College of Swansea, U.K.

7th World Conf. Earthquake Engineering, Istanbul, Turkey; September 1980. State of the Art in Earthquake Engineering, 1981.

Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, Missouri, April-May 1981. 3 volumes published by Univ. Missouri-Rolla.

In reviewing the advances reflected by the papers presented to this session, material has been drawn from these conferences and from various journals.

TOPIC 1 -- EVALUATION OF SOIL PROPERTIES

In-Situ Measurements

Recent developments in the techniques of in-situ measurement of wave velocities through soil deposits have been described in a considerable detail by Hoar and Stokoe (1978). The most commonly employed procedure is the cross-hole method. Three or more "receiver" boreholes are drilled and cased to the desired depth in a linear array with spacings on the order of 2 to 5 m. Each casing is grouted in place with cement in order to insure intimate contact and good coupling with the surrounding soil, and velocity transducers are installed to pick up the motions of the soil. To provide an impulsive source of energy, another "source" borehole must be drilled about a few meters distant from the nearest receiver borehole in the linear array. A standard penetration test (SPT) spoon is inserted in the source borehole to the same depth as the transducers, and an impulse is generated by dropping a weight onto the top of the drill rod. To this rod a vertical velocity transducer is attached to send a signal that triggers a storage oscilloscope. The body waves propagating through the soil deposit are monitored by the velocity transducer at each receiver hole and recorded on the same storage oscilloscope. After finishing the measurements at a certain depth, the velocity transducers at each receiver hole are lowered to the next depth and the source borehole advanced to that depth. The procedure is repeated to obtain the wave velocity through the soil at this depth. When a vertical impulse is applied by means of the SPT equipment, both vertically polarized shear wave (SV-wave) and compressional wave (P-wave) are generated. These two waves can be monitored at the receiver boreholes using vertically oriented and horizontally oriented velocity transducers, respectively.

One of the points of interest in Paper 10/10 by *Hoar and Stokoe* is the use of a new mechanical wedging system to produce the impulse in the source borehole. This is called an in-hole source. In the field procedure, a hole is drilled first to the final depth and cased all the way down before testing. By means of the mechanical wedging system, a plug is clamped tight to the wall of the cased borehole. The impulse is applied to this plug through a rod which extends to the ground surface. One of the advantages of utilizing this in-hole source is that it is readily possible to apply both vertical impulse and torsional impulse. A vertical impulse produces a vertically polarized wave (SV-wave) and a compressional wave (P-wave), while the torsional impulse generates a horizontally polarized wave (SH-wave).

Hoar and Stokoe compare the results of the in-situ test using the SPT device and the mechanical wedging device as the sources to produce vertical impulse. It was shown that both sources can generate readily identifiable SV-waves, but the SPT device can generate larger and more distinct arrival signals for the compressional wave. One of the interesting features of using the mechanical wedging device is that it is possible to reverse the direction of torsional impulse. The in-situ tests in this vein showed that the use of the reversely polarized shear wave data greatly facilitates the identification of the arrival time on the storage oscilloscope.

Hoar and Stokoe suggested a method to determine the material damping of in-situ soil deposits by analysing the wave propagation data. A comparison of the material damping obtained from the field measurements and from the laboratory resonant column tests on undisturbed specimens showed that the in-situ measured values of the material damping tend to be larger than the laboratory values by about a factor of two.

A new development in the technique of the cross-hole seismic method is presented by *Rodrigues* (Paper 10/24) who employed a new shear wave hammer to generate a vertically polarized shear wave. This hammer system uses a hydraulic pump to expand two plates for clamping the source to the borehole wall. The percussion system to produce an impulse is a cable-operated weight. The impulse is produced by dropping the weight to this source in the borehole. One of the novel features of this device is that it is possible to apply impacts in upward direction as well as in the downward direction. By performing a pair of tests involving reversal of the impact direction, the shear wave arrival can easily be distinguished on the display screen of the storage oscilloscope. The above technique has been successfully applied to several deposits at construction sites in Portugal.

Resonant Column Tests

The resonant column device has been widely in use to measure the shear modulus of soils in the small strain range in the laboratory. Reviews of the development of this testing method up to 1977 were presented in the State-of-the-Art report at the time of the Tokyo Conference. Since then, some progress has been made as described by papers by Iwasaki et al. (1977) and Kokusho (1980).

However, most of these works have been limited to soils having maximum particle size on the order of 5.0 mm. For very coarse grained soils involving gravel, crushed stone and cobbles, there has been practically no data concerning shear modulus because of the difficulty in constructing a large-scale resonant column testing machine. *Prange* (Paper 10/22) provides an interesting breakthrough to this difficulty. He constructed an apparatus accommodating cylindrical samples 1.0 m in diameter and 2.0 m in height. The resonant column sample is of the fixed-free type with electro-dynamic vibrators providing a torsional moment at the top. A ballast material for railway track, having a mean diameter of D_{50} = 45 mm, was tested by this apparatus. Based on the results of tests employing different confining stress, $\bar{\sigma}_0$, and different void ratio, e, a relationship was sought between the shear modulus, confining stress and void ratio. It was concluded that for the infinitesimal strain range on the order of 10^{-5}, the shear modulus of the ballast can be described by the equation

$$G_0 = \frac{7230(2.97 - e)^2}{1 + e} \bar{\sigma}_0^{0.38} \quad (\text{in KN/m}^2)$$

This is an equation similar to that discussed by Hardin (1978). It is of interest to note that the effect of void ratio can be expressed by the same function as for angular sands and also that the effect of the confining stress enters into the formula in the same fashion as in the formula for sands at comparable small strains.

The damping ratio of the ballast measured by this resonant column apparatus showed rather larger values, as compared to the damping ratios measured thus far for sands. This deviation was attributed partly to the friction inherent to the apparatus.

Because of the difficulty in applying strong forces to the large specimen with high frequencies, it was impossible to produce torsional strains in the specimen exceeding 2×10^{-5}. Therefore, it would appear difficult to obtain a complete picture of strain-dependent shear modulus and damping curves with the aid of this apparatus.

It is a well-established fact that in-situ deposition of soils generally results in the arrangements of particles such that the deposit acquires a cross-anisotropy with the axis of symmetry oriented in the vertical direction. The anisotropy, exhibited in the shear modulus and damping characteristics of a soil deposit, is one of the most important aspects in relation to the dynamic response analysis of the deposit when subjected to shaking during earthquakes. Very few studies appear to have been made thus far for this feature of the problem. *Bianchini and Saada* (Paper 10/3) performed an interesting series of laboratory resonant column tests on clays to provide test data clarifying the anisotropic characteristics of shear modulus and damping of the clays. A resonant column apparatus was modified so that it could excite a hollow cylindrical specimen in the axial as well as in the torsional mode, and thus yield values of shear modulus in the vertical and horizontal direction for the same specimen. A kaolinite clay and an illitic clay were consolidated with K_0-conditions to obtain anisotropic specimens. Isotropically consolidated specimens were also provided. The specimens had an inner diameter of 5.1 cm, an outer diameter of 7.1 cm, and a length of about 12.5 cm. The results of the resonant column tests showed that the values of the shear modulus, at the strain range between 0.15×10^{-5} and 2×10^{-5} at either mode, can be characteristically described by

$$G_0 = H \frac{(2.97 - e)^2}{1 + e} \bar{\sigma}_0^{0.5}$$

where H is a constant to be determined in the shear or the axial mode. For the Kaolinite, H differed by only about 10% for the two types of consolidation. The authors also studied modulus and damping at large strains.

Cyclic Load Tests

For the structural design of road pavements, permanent deformation characteristics of subgrade soils are of prime concern. *Farrell and Kirwan* (Paper 10/5) present a paper reporting the results of a series of repeated loading triaxial tests performed on glacial till materials. The influence of stress level, stress history and number of load application on the permanent deformation of compacted samples was studied, and a single parameter termed the creep compliance was suggested as an index for representing the permanent deformation characteristics of soils. The values of the creep compliance as obtained from the laboratory sample compared favorably to those obtained in a laboratory pavement simulator designed by the authors.

TOPIC 2 -- MACHINE FOUNDATIONS

From the standpoint of the geotechnical engineer, the design of machine foundations generally means estimating the natural frequencies for a proposed machine-foundation system and also evaluating the dynamic motions expected to occur at the operating fequencies. This means carrying out some form of dynamic analysis. The question of how such an analysis can and should be made, and how the associated parameters are to be evaluated, has been the topic of considerable research and discussion for the past half-century. While the major advances were made some time ago and have been adequately presented in books, state-of-the-art papers and in the General Report to IX ICSMFE in Tokyo, research and discussion continues today.

Historically, the analysis usually employs a mathematical model of a rigid block supported by an arrangement of springs and dampers representing the resistance of the soil -- and of piles should any be present. This type of analysis is still predominant for design studies, because it can be carried out readily and inexpensively. More sophisticated mathematical analysis -- using finite element models or the theories for a visco-elastic half-space or layered-space -- have played and still play crucial roles in guiding the formulation of appropriate block-spring-dashpot models and in the interpretation of tests. In some very special and expensive projects, these more sophisticated methods may be appropriately used in design.

Analysis of a machine foundation is a special case of the general problem of dynamic soil-structure interaction. Other examples are soil-structure interaction during earthquakes and the analysis of foundations for off-shore structures. Many of the recent developments in these latter areas are of potential value to the machine foundation problem, even though the scale of the foundations may be quite different. This is true of predicting permanent movements of foundations as well as evaluating dynamic response.

Theoretical Methods

Recent developments with regard to theoretical methods have concentrated upon the effects of layering within the soil, embedment of the foundation and interaction among adjacent structures.

Layered Soil: *Kagawa et al.* (Paper 10/16) present results from a parametric study for the case of a relatively thin stratum overlying a second very deep stratum -- a situation simulating a stratum over a half-space. A vertical periodic load is applied to a rigid circular disk at the surface. A finite element approach was used, employing the treatment of lateral boundaries developed by Waas. The paper does not state just how far away the lateral boundaries were placed, but a discussion of attenuation (see subsequent topic of Vibrations) would imply they were at considerable distance. The ratio of the thickness of the upper strata to the diameter of the foundation was varied, as was the ratio of the shear modulus in the two strata. Curves are presented for the amplitude and frequency of maximum magnification. The authors conclude that the lower stratum becomes effectively rigid when its shear modulus exceeds 20 times the modulus of the surficial stratum, although the basis of this conclusion is not clear in the results presented in the paper. An equivalent 1-degree-of-freedom model is suggested, using the static stiffness for the layered system and a damping coefficient which varies with the thickness and stiffness ratios.

Iljichev et al. (Paper 10/12) outline a different method for analyzing a layered soil. It employs an impulse transfer function -- which gives the vertical displacement of a weightless foundation subjected to a single instantaneous impulse. Two examples are presented, one of which involves an actual situation for which test data were obtained. In both examples, the response for the layered situation is compared with that for a half-space having the properties of the top-most layer. The layering does not have a major effect in either case. An approximate representation of the impulse transfer function is mentioned, but the paper does not indicate how the parameters should be determined for the general case.

The effect of stratification within the soil has been discussed extensively in the literature of the 1970s. The general phenomena associated with a softer stratum overlying a stiffer half-space are well known: the stratum has natural frequencies and at low frequencies radiation damping is reduced greatly. These phenomena can lead to significantly amplified motions unless care is taken in the design of a foundation. Simple procedures for evaluating approximately the soil-foundation stiffness functions have been suggested (see references to work of Kausel). One question of practical importance is: how deep must a stratum be before the effect of an underlying stiff stratum is inconsequential. From the various studies, the following rules are suggested:

Vertical excitation: $H > 7\ D$

Horizontal excitation: $H > 2.5\ D$

Rocking excitation: $H > 1\ D$

where H is the depth of the stratum and D is the diameter of the Foundation. The rule for torsional excitation should be similar to that for horizontal and rocking excitation.

It would be desirable to use the available theoretical methods to develop more general guidance for purposes of design. Doing so is made difficult, of course, by the infinite variety of possible stratifications.

Embedment: As discussed in the State-of-the Art Report to IX ICSMFE, the work of Novak is most commonly used to evaluate the influence of embedment upon teh stiffness of a foundation. Other studies have been made using the finite element method. For example, Kausel in several papers has proposed the following factors (Kausel et al. 1978; Kausel and Ushijima 1979):

Vertical stiffness: $1 + \frac{2}{3}\frac{E}{R}$

Horizontal stiffness: $1 + 0.5\ \frac{E}{R}$

Rocking stiffness: $1 + 2\ \frac{E}{R}$

Torsional stiffness: $1 + 2.67\ \frac{E}{R}$

where E is the depth of embedment and R is the radius of the foundation. The effective location of the horizontal reaction is at a height $(0.4E - 0.03R)$ above the bottom of the foundation. These formulas apply for $E/R < 1$ and for half-space conditions. (Corresponding factors have also been proposed for the case where the surficial soil is underlain by a stiffer earth material). Full contact is assumed between the sides of the foundation and the surrounding soil. Both the real and imaginary parts of the stiffness functions are multiplied by these factors, so that damping ratios increase along with (actually even more than) the spring constants.

One paper to this session deals with embedment: *Ranjan et al.* (Paper 10/23). The authors present the equations of motion for an embedded block in terms of subgrade modulus, including moduli for vertical compression at the base, horizontal compression at the sides, horizontal shear at the base, vertical shear at the sides, and rocking. Dynamic experiments, using blocks 1.5 m by 0.75 m, are interpreted to study trends in the subgrade moduli as a function of the depth of embedment. Equations are presented for the variation of effective vertical subgrade modulus with embedment, and for the associated effective mass and damping ratio. The ratio between rocking and horizontal subgrade moduli is said to remain substantially the same (ratio $\simeq$ 3.5) for all embedments, and a curve giving the increase of these moduli with embedment is given. Owing to the limitation on the length of papers, *Ranjan et al.* could not present the data on which these conclusions were based, and until a fuller study is possible the proposed equations and curves should be used with caution. The authors again emphasize the importance of contact between the soil and the sides of the foundation.

Interaction among adjacent foundations: This topic is important when evaluating the dynamic motions caused by multiple machines and in selecting the required spacing between adjacent machines. It is also much-discussed in connection with the analysis of closely-spaced structures subjected to earthquake ground shaking, a situation which typifies nuclear power plants (e.g. Murakami and Luco, 1977). The same topic is encountered in the study of ground interaction with high-speed rail systems.

Two papers to Session 10 address the theoretical analysis of this problem for masses resting on the surface of an elastic half-space: *Sarfeld et al.* (Paper 10/25) and *Holzlöhnen* (Paper 10/11). The authors outline the governing equations and boundary conditons and suggest different techniques for the solution of these equations, provide some illustrative results and explore possible simplifications and approximations. While the paper by Sarfeld et al. is specifically oriented toward the analysis of rail systems, it provides results and conclusions of general interest.

Because of the complexity of the problem, the search for approximations and implications is of particular importance. One interesting question is: How much is the dynamic response of one mass affected by the presence of nearby rigid but massless plates? *Holzlöhner* addresses this question and concludes: Not significantly, at least when the spacing between foundations equals or exceeds the width of the foundations. A second question is: Can the motions at a nearly rigid plate be approximated satisfactorily by the surface motions calculated ignoring this rigidity? *Holzlöhner* says the answer is yes. These same conclusions are reached (or implied) by *Sarfeld et al.* Taken together, these conclusions indicate that enormous simplifications can be made in many cases.

A different question is: How does the response of a single mass change when other masses are placed nearby? Here the answer is: The effect can be quite significant. For example, consider a symmetrical foundation subjected to a vertical dynamic load acting through the center of gravity. Alone this foundation will experience only vertical motion. Surrounded by other masses -- even masses that are not acted upon by dynamic loads -- the foundation may rock and sway significantly. This was illustrated vividly in the work of Warburton et al. (1971), although the effect is exaggerated when damping within the half-space is ignored. *Sarfeld et al.* explore the question: If there is an infinite series of masses located along a line, how many of these masses must be included when determining the response of one mass to a dynamic load? The answer is: At least 3 masses on either side. If the sub-soil is a stratum, the interaction effects may be even more important because the number and spacing of the masses will have a strong influence upon the natural frequencies of the system (see, for example, Gonzalez and Roesset, 1977).

Still another important question is: How well can a 3-dimensional problem be approximated by a 2-dimensional analysis. *Sarfeld et al.* investigate this point, and find that a 2-dimensional analysis greatly overestimates interaction between adjacent square masses. (This conclusion may have great import to the analysis of nuclear plants.) Conversely, the analysis of adjacent square masses underestimates severely the interaction between long rectangular masses such as railroad ties.

The topic of interaction among adjacent structures is ripe for further study. The goal of the sophisticated analyses which can be formulated and carried out on modern computers should be to develop general principles and rules that can be used in practical design. Given the complexity of the problem, this is a challenging task.

Evaluation of Design Parameters

The evaluation of the stiffness of the soil is critical to the prediction of the performance of a machine foundation. This is true whether the engineer is selecting a value of shear modulus to use in an equation from the theory of elasticity or choosing values of subgrade modulus. Evaluation of damping may be important in some cases. While considerable attention is often given to evaluation of an effective mass of soil, this effort is justified only if careful attention has been paid to the choice of an equivalent stiffness.

Two papers to this session (*Yan*, Paper 10/32) *Sreekantiah*, Paper 10/28) discuss the evaluation of coupled rocking/swaying parameters directly from tests upon small (~ 1-2 m) blocks. The limitation upon the length of papers has forced the authors to omit many details concerning these tests and concerning the method for extracting parameters from the measurements. Hence the General Reporter finds it difficult to comment in detail upon these papers. It would appear that little attention has been given to two considerations which can have a major influence upon the choice of parameters for design of an actual foundation: the amplitude of dynamic strain and the level of initial effective stress within the soil. For example, in both tests the dynamic motions are large compared to those which would usually be acceptable in an actual foundation. These points are discussed in Papers 10/2 and 10/21, which are reviewed subsequently under the heading of Case Studies.

One aspect of *Sreekantiah's* paper is quite interesting. Tests were made upon a number of blocks of different size at the same site. The parameters evaluated from one test were used to predict the motions of the other blocks. (Three of the blocks were individually used in this way as reference tests.) The errors between predicted and observed motions ranged from 1% to 125%, with a mean of 45%. These results are a good indication of our ability to predict the motions of a machine foundation. Without the benefit of actual tests at a site, we should not expect to do as well as this.

Case Studies

Every possible opportunity must be taken to compare predictions with measurements made upon actual foundations. Such case studies provide checks upon the validity of theories, although usually they really are tests of the methods for evaluating the parameters appearing in the theories. Two comparisons between predictions and observations are presented in the papers to this session.

A direct foundation: *Prakash and Puri* (Paper 10/21) discuss an irregularly shaped foundation -- about 3 m across -- for a reciprocating compressor which applied an off-center vertical load at 6.75 Hz. The sub-soils were sands and silty sands, with blow counts of about 8 just below the foundation and 15 by 6 m below the block. The block was founded 2.4 m below ground surface, but was surrounded by an air-gap. The observed dynamic motions were excessive: about 0.32 mm horizontal motion at the top of the block, compared to a permissible motion of 0.0125 mm.

The response of this foundation was computed (after the fact) using the equations for a lumped mass-spring system with uncoupled vertical motion plus coupled rocking and swaying. The spring constants were evaluated by two methods: (a) Barkan's method, employing semi-theoretical ratios among the subgrade modulus, and (b) formulas from the theory for a uniform elastic half-space, involving shear modulus and Poisson's ratio. When computing dynamic motions using "Barkan's spring constants", damping was ignored. Radiation, but not internal damping, was included when employing the "half-space spring constants". The effective mass of the soil was not included in either case. Actually, the differences between the two methods were really of little practical significance, inasmuch as the vertical sub-grade modulus for "Barkan's method" was computed from the shear modulus used in the "half-space method".

Of greatest interest in this paper are the techniques used to evaluate the shear modulus of the soil. Vibration tests on small blocks were conducted, as were cyclic plate loading tests. The shear moduli deduced from these tests were corrected for level of strain and vertical effective stress, so as to arrive at a value applicable to the actual foundation. These procedures appear to be sound and useful.

Based on this modulus, the fundamental frequencies computed by the two methods bracketed closely the operating frequency. Since, in view of the very large observed motions, it seems reasonable to presume that the foundation was at a near resonance, it may be said that the computed results agree with observation. The computed motions were within a factor of 1.2 to 2.8 of the observed motions, which is remarkable considering that a lightly- to moderately-damped system was somewhere close to resonance. These results confirm that the choice of the shear modulus was substantially correct. Certainly, if such an analysis had been made at the time when the foundation was designed, it would have correctly predicted that motions would be excessive.

It is difficult, in any such case study, to reach more definite conclusions unless a foundation can be excited at different frequencies so as to determine the actual resonant frequency -- which takes very special machinery. Such information is necessary if a detailed comparison of observed and computed motions is to be meaningful. Lacking the opportunity to vary the frequency, a record of the phase difference between applied load and resulting motion can be useful.

A pile foundation: Paper 10/2 by *Bagchi* deals with a large foundation (approximately 10 m × 12 m) for a highly unbalanced reciprocating compressor. The subsoil consists of a sandy, silty clay to a depth of about 3 m, followed by a meter of fine medium sand and then a medium to fine dense sand to considerable depth. In this latter material, blow counts ranged from 30 to 45 blows/ft. The mat was supported on bored, cast-in-place piles with a length of 10 m.

During the initial stages of design, it was assumed that the piles were end-bearing as regards vertical stiffness. Horizontal stiffness was evaluated by assuming the effective length of pile to be 2.4 m. Apparently the vertical and horizontal stiffness were then computed using only the material properties for the pile.

As construction began, dynamic tests were made upon a single pile. It was found that the horizontal stiffness coefficient (units of subgrade modulus) was quite sensitive to the amplitude of vibration, whereas the vertical stiffness coefficient decreased only slightly with amplitude. These are significant results which are in accord with the general experience concerning piled foundations.

The test results validated the values of stiffness used for preliminary design. The actual natural frequencies of the completed foundation was measured, although very few details of these measurements are presented. The observed frequencies agreed very well with those predicted on the basis on the dynamic tests on a single pile, and the measured amplitudes at the operating frequency (about 0.7 of the lowest observed natural frequency) also agreed well with the predicted amplitudes.

The author emphasizes the importance of evaluating the horizontal stiffness from dynamic tests on an actual pile at the site, as a function of the amplitude of vibration. The General Reporter agrees that this is a prerequisite for a confident prediction. The author also suggests that the behavior of driven and bored piles is similar.

Correcting Vibrations

When excessive vibrations are observed or are predicted by analysis, the foundation engineer usually thinks in terms of geotechnical solutions: changing the size of the foundation, providing underpinning or piling, stiffening the soil, etc. However, one should always be alert to the possibility of corrective action that does not rely safely on soil mechanics. When rocking is the problem, lowering the foundation or better centering of vertical forces may achieve the necessary reduction in vibrations. Two other approaches are discussed by *Iljichev et al.* (Paper 10/12). The use of dampers on the machine is a passive approach. An active approach employs some means for putting controlled additional energy into the machine at such times as to interrupt the vibration pattern, thus suppressing the build-up of excessive vibrations. Theory is outlined and examples are presented to demonstrate the feasibility and potential benefits of both approaches.

Settlement

It is well known that machine foundations over loose sands, especially saturated sands, can experience excessive settlements. This possibility should always be checked before an analysis for dynamic response is undertaken.

Eisler et al. (Paper 10/4) give a brief description of a thorough study of the foundation for a turbo-generator. Finite element analyses were used to determine the spatial variation of dynamic stresses within the soil. The effects of initial stresses and dynamic strain amplitudes were taken into account in these analyses. Triaxial and direct shear tests were employed to establish the dynamic stresses which would cause residual volumetric strains. In this case it was concluded that the foundation was safe from the standpoint of excessive settlements. This example illustrates the type of study which is possible and desirable for important projects.

Aubry et al. (Paper 10/1) also touch upon the topic of settlements, although they deal with settlements caused by earthquake type shaking. Their results show that horizontal shaking causes considerably more densification than vertical shaking (at 0.08 g acceleration), although vertical accelerations are significant when combined with horizontal. This study confirms previous conclusions by Youd (1972), Seed and Silver (1972) and Pyke et al. (1975).

TOPIC 3 -- DAMAGE BY VIBRATIONS

This topic is typically subdivided into two sub-topics: (a) attenuation of vibrations as they pass through the soil, and reduction of these vibrations by screens: and (b) the levels of vibration that may be damaging to buildings. There is one excellent paper to this session concerning each sub-topic.

Attenuation and Screening of Waves

Haupt (Paper 10/9) presents and interprets data obtained from tests in a special bin of soil, having a diameter of 8.3 m and a depth of 3 m. A small exciter was used to create the vibrations and special transducers were developed to record the motions with minimal disturbance to the field of motion. Screening by solid obstacles, rows of boreholes, sheet pile walls and open trenches were all under investigation. Careful comparisons were made with results from finite element studies (described during IX ICSMFE) and with experimental results from earlier investigators. In general, the results have once again confirmed that the key variable is the size of the screen relative to the Rayleigh wave length. Some of the more specific conclusions are:

* The screening effect of solid obstacles depends upon their cross-sectional area normalized by the square of the wave length, and does not depend upon their actual shape. This result had earlier been predicted by theory.
* Sheet pile walls are relatively ineffective, at least for wave lengths on the order of the depth of the piling.
* Theories proposed for the screening effects of open trenches are good, at least for the region immediately behind the trench.

The experiments with boreholes were less conclusive because of the difficulty in maintaining open holes. The author recommends further study of this technique. This paper is recommended for close study by any engineer interested in the use of screening.

Attenuation as predicted by theory is mentioned briefly by *Kagawa et al.* (Paper 10/16). The attenuation of motion w along the surface away from the structure was evaluated, and found to fit the standard equation combining both geometrical dispersion of Rayleigh waves and internal damping:

$$w \sim \sqrt{1/r}\ e^{-\alpha r}$$

Effects on Structures

Studer and Suesstrunk (Paper 10/29) report upon the Swiss guidelines aimed at preventing damage to buildings from vibrations. These guidelines set forth limiting velocities, as measured at foundation level, as a function of frequency, class of construction, and source of vibrations. At the lower end of the frequency range (10-30 Hz), the limiting values are, depending upon the class of construction:

For vibrations from blasting: 8 to 30 mm/s

For vibrations from machines or construction equipment: 3 to 12 mm/s

Higher velocities are permitted above 60 Hz for blasting and above 30 Hz for machines and construction equipment. The velocities are not varied for soil conditions, but soils will influence the amplitude and frequency of incoming vibrations.

These guidelines are based upon 200 investigations. The chosen values are a compromise reflecting the diverse viewpoints of the members of the cognizant committee. The velocities are higher than previously permitted, but lower than thought justified by the authors. It had been the intent of the committee to include in the standard a procedure for predicting the ground motions resulting from blasting, but it developed that there were too many variables and uncertainties. The standard recommends that measurements be made whenever there is concern, and outline a suggested procedure.

TOPIC 4 -- DEFORMATION AND STRENGTH DURING CYCLIC LOADING

Definition of Failure

In order to obtain in-depth understandings of the deformation characteristics of soils under seismic loading conditions, carefully conducted triaxial tests involving slow unloading and reloading are preferred to tests performed with higher frequencies of 1 to 3 Hz. The laboratory cyclic triaxial test results in this vein are reported by *Luong and Sidaner* (paper 10/18). It was found convenient to introduce a threshold state of stress separating the contractive and dilative behavior of sand, in order to provide meaningful interpretation of the deformation characteristics. This threshold state of stress is defined by a straight line emanating from the origin and being subtended by the failure line in the p'-q stress space, where p' is the effective mean principal stress and q is the deviator stress. This characteristic line in the p'-q stress space corresponds to a state of stress where no volume change can occur in the drained test. Whenever stresses are changed in the domain below this line, the deformation of sand is stable and contractive, and there is no hysteresis of volume change if unload-reload cycles are executed within this domain. In contrast to this, if stresses are changed across or above the characteristic line, buildup of permanent displacement or rupture was observed to occur in the sample. It would appear that the characteristic line mentioned in *Luong and Sidaner*'s paper is similar to what is referred to as "line of phase transformation" that was used in the paper by Ishihara et al. (1975).

Wang's Paper 10/31 is also concerned with the definition of failure of saturated sands during cyclic loading conditions. Effects of initial shear stress on liquefaction-type failure of sand are considered on the effective stress basis. It was shown that application of an initial shear stress in drained conditions leads to an increase in resistance of saturated sand in subsequent undrained cyclic loading. Failure conditions relevant to the state of stress are proposed by considereing the Mohr-Coulomb type failure criteria in terms of effective stress. A brief reference to the collapse of loess soils during earthquake shaking is made by *Ivanov et al.* (Paper 10/14).

Multi-Directional Shear

The behavior of sand subjected to multi-directional shear on a fixed plane is of interest because it lends to the clarification of liquefaction resistance of horizontal sand deposits during earthquakes undergoing cyclic shear stresses that change not only in amplitude but also in direction. This aspect of the problem was investigated recently by Pyke et al. (1975), who conducted multi-directional shaking table tests in which a layer of dry sand in a small shaking table was mounted transversely on another large shaking table. Based on these test results, Seed et al. (1978) calculated the resistance to liquefaction of the same sand that would have been obtained in a simple shear type stress condition if the sand had been saturated and subjected to the cyclic shear stress executed in a gyratory manner. It was found that the resistance to liquefaction under 20 cycles of uniform gyratory shear stress is about 15% smaller than the liquefaction resistance that would be obtained in uni-directional loading. More recently, Casagrande and Rendon (1978) designed and constructed a simple shear test apparatus in which cyclic shear stress could be applied in multi-directional loading conditions. Another study in similar vein was conducted by Ishihara and Yamazaki (1980) using a multi-directional simple shear test apparatus equipped with loading rams positioned in two mutually perpendicular directions. In one series tests on loose saturated sand specimens employing circular and elliptic load paths with a phase difference of 90 degrees between stresses in mutually perpendicular directions, it was shown that the cyclic stress ratio inducing 3% simple shear strain under a given number of cycles decreased as the amplitude of the second component was gradually increased. When the second component grew as large as the main component, the cyclic stress ratio dropped to approximately 65% of the cyclic stress ratio causing 3% strain under uni-directional loading condition. *Ishihara and Yamada* (Paper 10/13) presented results in similar vein from tests on sand using a true triaxial apparatus. The octahedral plane within a cubic specimen was envisaged as a plane on which shear stresses would be varied in direction as well as in amplitude, and the three principal stresses were cyclically changed so that their combination could induce rotational and crisscrossing shear stress paths on this plane. The test results also showed a reduction in cyclic stress ratio causing liquefaction in a given number of cycles, as the configuration of the stress path on the octahedral plane changed from a straight line to an ellipse and further to a circle.

Effect of K_o

Schwab (Paper 10/26) investigated the effects of initial K_o-conditions on the failure of saturated cohesionless soils under cyclic loading conditions. A usual type of the triaxial cell was equipped with a horizontal rod through the flank of the cell to apply cyclic horizontal loads to the mid-height of a cylindrical specimen 30 cm in diameter and 50 cm long. In the mid-height, a disk-shaped perforated plate was inserted horizontally in the specimen and a ring was attached to the periphery of the plate. The horizontal loading rod was connected to this ring to transfer the horizontal cyclic load to the specimen. With the top and bottom ends of the specimen fixed horizontally, it was thus possible to produce simple shear type deformation in the upper half and lower half of the specimen under any desired K_o-conditions. At both ends of the specimen, a drainage control device was installed to permit cyclic simple shear tests under controlled drainage conditions.

In one series of tests in which K_o was maintained throughout cyclic undrained simple shear, amplitudes of horizontal shear strain and axial shear strain were measured as functions of the number of cycles. It was noted that the amplitude of the horizontal shear strain relative to that of the axial shear strain at a certain number of load cycles depends upon the K_o-value imposed on the specimen. The lesser the K_o-value, the larger the axial shear strain amplitude as against the horizontal shear strain amplitude. Failure of the specimen was considered to have occurred when 5% single-amplitude strain was induced either in the horizontal or vertical mode of shear strain. In the type of the cyclic shear test as above, two components of shear strain are involved and it appears necessary to consider two strain components, in combination or separately, to define any failure criteria. This appears to be a new area in which further studies are necessary.

Dynamic Effective Stress Method

During the last several years there have been growing interest in the development of a dynamic effective stress method for the analysis of ground response during earthquakes and in connection with off-shore structures. The effective stress method of analysis consists of solving the non-linear equations of motion for the ground coupled with generation and dissipation of pore water pressures, with resulting time change in soil properties. A key element in this method of analysis is the establishment of a proper material model in order to predict increments of pore water pressures that occur during various time increments of shear stress in the course of irregular seismic excitation.

Several pore water pressure models have been proposed and an excellent review was presented by Finn (1979).

Paper 10/6 by *Finn and Bhatia* proposed a practical approach to pore water pressure prediction guided by the intrinsic time concept of endochronic theory. The pore water pressures are expressed directly as a monotonically increasing function of a single variable called a damage parameter. The new parameter was defined as a function of the current value of shear strain and the strain history experienced by the sand. It was proposed that the strain history be represented by the length of the strain path. In the case where test data are available in terms of shear stress instead of shear strain, a similar damage parameter was found to be defined in terms of the current value of shear stress and the length of the stress path. The proposed concept was substantiated by simple shear tests performed on seven clean sands.

The dynamic effective stress approaches have been proposed exclusively for cohesionless soils in relation to prediction of liquefaction in sand deposits. One factor accounting for the particular emphasis placed on cohesionless soils appears to be the ease with which theories can be tested and verified in laboratory tests on clean sands. In contrast to this, interpretation of cyclic tests on cohesive soils on the effective stress basis has been considered a prohibitively difficult task, because of uncertainty involved in accurate measurements of pore water pressures in laboratory test specimens. A successful effort was made by *Matsui and Abe* (Paper 10/19) to measure pore water pressures in triaxial specimens of clay by embedding a miniature piezometer in the middle of the specimens. In undrained cyclic isotropic loading tests (in which pure compressional stress is applied repetitively) on saturated clay, the cyclic variation of pore water pressure measured by the miniature transducer showed a complete coincidence, both in amplitude and in phase, with the cyclic change in the externally applied isotropic stress. This observation indicates that the pore water pressure produced by the undrained isotropic loading was perfectly picked up by the transducer imbedded in the specimen. Guided by this success, *Matsui and Abe* were able to trace the effective stress paths for the clay sample under cyclic loading conditions. A feature of the test results deserving attention is the fact that the pore water pressure could become momentarily equal to the initial consolidation pressure after a certain number of cyclic shear stress had been applied to the clay specimen. This is a state of stress very much similar to the state of cyclic mobility defined for dense sand by Castro (1975).

Particle Shape and Packing

It has been known that the resistance of saturated sand to liquefaction depends upon the state of packing as typically represented by the relative density. However, recent studies by Ladd (1974) and Mulilis et al. (1977) showed that, if sand is deposited through different methods, different fabrics are formed in the packing and can exert equally important influence on its cyclic behavior even when the relative density is the same. Therefore, it now becomes apparent that the use of the relative density is meaningful only when comparing the cyclic behavior of different sands compacted to different densities under an identical placement condition. Even for this modified interpretation for the proper use of the relative density, however, some skepticism has been raised. Ishihara and Watanabe (1976) showed, for example, that, even if the relative density is identical, a greater cyclic resistance can be mobilized for relatively uniform coarse-grained sands. This kind of sand generally has a small difference between the maximum and minimum void ratios and, consequently, has a smaller potential to decrease volume from a given state of packing under cylcic loading conditions. In cognizance of this fact, the use of the volume decrease potential defined by $e-e_{min}$ was suggested as a better parameter than the relative density to represent a state of packing of sand deposited under similar conditions, where e is the void ratio and e_{min} denotes the minimum void ratio.

Ivanov et al. (Paper 10/14) presented an interesting paper in this vein, suggesting the use of shape factor as a new parameter to identify physical properties of sand. They measured, using a microscope, shapes of a hundred particles each for five different sands having nearly identical grain size distributions. By analyzing statistically roundness and sphericity of particles, a parameter called shape coefficient was determined for each of the five sands. Among the five sands investigated, the shape coefficient, K, took a largest value of 0.38 for Shulbin sand consisting of round-shaped particles and a smallest value of 0.12 for Krasnorechenskij tailings sand composed of angular-shaped particles. The authors of this paper present interesting tests on the influences of the shape coefficient on some physical properties of the sands. First of all, it was shown that, under an identical void ratio of 0.67, the Shulbin sand with the highest value of the shape coefficient exhibited a permeability coefficient as large as 4.0×10^{-2}, as against a permeability coefficient of 5×10^{-3} obtained for the tailings sand having the lowest shape coefficient. The effect of the shape coefficient on the angle of internal friction was shown to be also significant, with the result that, under their minimum void ratio conditions, the Shulbin sand shows an angle of internal friction of 36°, as contrasted to a larger angle of internal friction of 46° measured for the most angular-shaped tailings sand. Unfortunately, the effect of the shape coefficient on liquefaction resistance of sand is not presented in this paper in a manner directly comparable to the results of other investigations.

Sea Bottom Soils

In numerous zones of interest to oil industry, sea bottoms are covered with carbonate sediments which are mixtures of carbonates and silicates. Their grains are brittle and can break under a stress of only a few 100 Kpa. Angularity of grains is generally pronounced and cementation is randomly dispersed in the skeleton of the material. For the purpose of clarifying the mechanical characteristics of these special soils, several kinds of laboratory tests were performed by *Nauroy and Tirant* (Paper 10/20) on undisturbed specimens of three different kinds of sand: algal sand, shell sand and detritic sand. The physical property tests showed that the behavior of the carbonate sediments does not differ from that of classical silicate materials, except for the compressibility index which varied from 0.02 for detritic sands (similar to silicate sand) up to 2 and even higher for algal sands. Oedometer test results have shown that the fraction of $CaCO_3$ in each gradation range has an important effect on the compressibility. A high percentage in large particles was indicative of biologically-evolved grains and accordingly of a high compressibility. On the other hand, a large percentage of $CaCO_3$ in the smaller grains yields a reduced compressibility due to the small-adsorption capacity of calcite. Drained triaxial tests showed a sharp decrease in the peak friction angle with increasing confining stresses due to grain breakage and hence a limited tendency to dilatancy. Cyclic triaxial tests were also performed on these materials, simulating wave loading on off-shore foundations. The results of tests were interpreted on the basis of the concept of the characteristic curve proposed by *Luong and Sidaner* (Paper 10/18).

With an eye to reproducing the state of stress in the soil element beneath the foundation of off-shore platform experiencing cyclic changes caused by wave motions, *Jessberger and Jordan* (Paper 10/15) manufactured a cyclic triaxial test apparatus in which the lateral stress could be changed cyclically as well as the vertical stress. By making the cell pressure change 180 degrees out of phase with the vertical stress, it was possible to produce completely reversing cyclic stresses on the 45° plane within the test specimen without creating any cyclic change in the compressional stress. Tests were performed on saturated medium dense sand specimens (relative density = 75%). Under two-way loading conditions in which the cyclic stress completely reverses its direction, the cyclic stress ratio causing 3% double amplitude strain in any given number of cycles was about half the similarly defined cyclic stress ratio executed under one-way loading conditions. In another series of tests, effects of drainage on the cyclic resistance of the sand were investigated. After specimens were subjected to a given number of cycles, a short period of drainage was introduced to reduce pore water pressures to some extent; then cyclic load application was continued until the specimens deformed to failure strains. It was shown that the drainage considerably increased the cyclic strength of the sand. In the case of cyclic loading such as that caused by the wave motions, the number of load cycles to be considered is well over 2000 and corresponding time duration is on the order of several hours. Therefore, there may be sufficient time for the pore water pressure to dissipate through relatively permeable sand deposits beneath the foundation of the off-shore structures. In view of this, the authors of this paper emphasize the importance of considering the effect of drainage on the cyclic strength of the sand when assessing the stability of the foundation of off-shore structures.

OTHER TOPICS

Wave Propagation

van der Kogel and Loon-Engels describe (Paper 10/17) a shock tube test device they developed for studying wave propagation through porous media. The tube had dimensions of 7.5 cm in diameter and 90 cm in length. In order to avoid undesirable friction between samples and inside wall of the shock tube, a thin layer of fluid about 1 mm thick was introduced. A total pressure gage and several pore pressure gages were embedded in the sample to monitor time-changes of pressures in the course of the compressional wave propagation. In the first series of tests using a dry sand, the total pressure gage responded clearly but there was no apparent sign of air pressure transmitted through the pores. Therefore, the compressional wave was considered to have propagated mainly through the soil skeleton which had much larger stiffness than the air in the pores. In the second series employing partially saturated sand, the pore pressure gages responded almost as clearly as the total pressure gage, which implies that the pore fluid as a whole had a stiffness as large as that of the soil skeleton. In the last series of the test using almost saturated sand, the response of the pore pressure gages was as pronounced as that of the total pressure gage. Although the test results presented in this paper are still qualitative, they were consistent with what could be predicted by a simple theoretical consideration.

Eisler et al. (Paper 10/4) discuss the propagation of shock waves emanating from a spherical explosion. Nearest the explosion is a zone where the soil is loosened, presumably because of delatency associated with large shear strains. Further from the explosion there is a zone of compaction. Another section of this paper discusses the difference between static and dynamic compressive modulus, which exists even in dry sand. It is suggested that there is a limiting rate of interpartical sliding. The relaxation time for sliding in sands is said to be 10^{-4} s.

Two-dimensional non-linear wave propagation is discussed by *Ivanov et al.* (Paper 10/14), specifically for the case of a weight falling upon a soft soil overlying rock. The compressive stress-strain curve for the soil is assumed S-shaped, so that there are elastic waves, plastic waves and -- at high stresses -- compaction waves with a propagation velocity greater than that of elastic waves. Reflections from the rock cause interactions among these waves, a number of which are illustrated by examples.

Earth Dams

During the four years since IX ICSMFE, there have been a series of major papers by Seed concerning the safety of earth dams during earthquakes. In the Rankine Lecture (Seed, 1979), the results of many years work concerning liquefaction in dams are summarized into a coherent theory. While stating clearly that the theories involve many incompletely tested assumptions and must be used with caution and judgement, Seed claims that such analysis can and should be used to help guide the design of new projects and the evaluation of existing dams. A paper by Seed et al. (1978) collects together experiences during earthquakes of many dams, principally in the United States and Japan. From examination of these data, several important conclusions are reached. There have been no serious failures of dams composed of soils which are not susceptible to rapid buildup of pore pressure during cyclic loads. Furthermore, even in dams made of sandy soils which are susceptible to liquefaction, significant failure has not occurred until the peak ground acceleration approaches 0.2 g. Makdisi and Seed (1978) have presented a very useful procedure for estimating the permanent deformation of dams whose strength is not reduced rapidly by cyclic loads. The method accounts for the influence of dynamic strain upon shear modulus and hence natural period, and is simple enough that calculations can be completed by hand in a few minutes. A similar procedure, which is especially useful where dams are founded over soil or soft rock, has been developed by Sarma (1979). The Comision Federal de Electricidad (1980) in Mexico has reported a thorough case study of the deformation of two dams during a major earthquake. The observed deformations were quite small.

All in all, this has been a significant four years for the designers of earth dams in seismic regions.

In X ICSMFE, *Gazetas and Abdel-Ghaffar* (Paper 10/7) outline the theory for the dynamic response of earthen dams in which the shear wave velocity varies with depth according to a power law. Results from measurements of dynamic motions at a number of dams are presented in support of this theory. Whereas the most common assumption of a uniform velocity leads to a quarter sine wave for the fundamental mode, the theory of the authors predicts a mode shape more like that for a cantilever beam. Thus a "whipping action" is predicted near the crest of the dam. The data from actual dams -- both measured shear velocities and measured dynamic motions -- do appear to support the contentions of the authors, although they do not explain how the variation of velocity with depth was measured. In other papers by the authors (see reference list for the paper) the theory is presented more completely, and is as easy to use in practice as the theory for uniform relativity. The General Reporter expects that the authors' theory will be used increasingly in the future.

Retaining Structures

The past four years has also seen a significant advance in the design of gravity retaining structures against earthquakes. This is the concept of designing those walls for deformation rather than for force (Richards and Elms, 1979). The principle is essentially that used for buildings: a structure may be designed for forces smaller than those predicted on the assumption of no yielding, provided that upon yielding the structure remains stable and does not distort excessively. Richards and Elms present a very simple procedure for implementing the approach, making use of the theory and results for a block sliding on a plane. The general validity of the method has been confirmed by model tests (Lai and Berrill, 1979), although these tests indicate that use of the sliding block analog is rather conservative. The conservatism can be accounted for by including the effect of the vertical accelerations which must occur when the backfill yields even though there is no vertical acceleration of the base (see Whitman, 1979). While further research is still necessary -- on the effect of tilting, for example -- this new concept will make the design of gravity walls much more rational than in the past.

In a paper to this conference, *Aubry et al.* (Paper 10/1) describe some results from model tests of walls subjected to base shaking. The use of rods to simulate a granular soil simplified the experimental arrangements; that is, there was no problem with side wall friction. The tests demonstrated the amplification of base accelerations vertically through the backfill, and that this amplification decreases with vertical acceleration. With a rigid wall, the observed dynamic lateral forces appeared very similar to those predicted by theory. The results as reported seem more qualitative than quantitative, but do serve to give confidence in the usefulness of the technique. Some results are also presented from an application of the finite element method to computing the response of an anchored bulkhead. However, since space did not permit a description of the assumptions used in the theory, it is not possible to interpret these results.

Soil-Structure Interaction During Earthquakes

At the time of IX ICSMFE, there was still a major debate underway among the proponents of the several methods -- finite elements, lumped springs, etc. -- for representing the effect underlying soil upon the dynamic response of buildings. Since then there has been a much clearer understanding of the similarities and differences between the various methods, and of the advantages and limitations of each. These matters have been discussed thoroughly in two major publications from the United States: Idriss (1979) and Johnson (1981). Both publications contain contributions by the principal workers in the United States.

Of course, there are still factors which are difficult to account for by analysis and whose effects hence are poorly understood. These major factors are the effects of adjacent structures and the nature of the input motion; i.e. body waves vs. surface waves, etc. These are topics for continuing research.

Another topic which as yet is poorly understood is the effects of non-linearities such as partial uplift of the foundation during earthquake shaking. *Taylor et al.* addresses this problem in Paper 10/30. They first develop a Winkler-type model of a bed of springs, and consider both separation and elasto-plastic yielding during cyclic rocking. Also reported are experiments using model foundations 0.5 m by 0.25 m. (Paper 10/1 by *Aubrey et al.* contains some additional experimental data concerning the stresses on the bottom of a rocking footing.) With clays, the theory predicts and the experiments show quite a different behavior depending upon whether the safety factor for vertical load alone is greater or less than about 2. When FS > 2 (or maybe 3) yielding occurs only in the first cycle and the settlement stabilizes. For FS < 2, yielding continues during each successive cycle and settlements accumulate. With sands, a progressive type of settlement appears for all safety factors, with the rate of settlement increasing as the safety factor decreases. These various results are basically the same as those previously noted by Rowe et al. (1976) in centrifuge tests and Zienkievicz and colleagues at Swansea in theoretical work. A name has been given to this phenomenon: "shakedown". Taylor et al. have several practical suggestions from their work, among them: the concept of "protecting" a column by having the soil "hinge" before yielding can occur in the column.

Paper 10/27 by *Sigismon et al.* deals in the modelling of a pile foundation for a nuclear power plant. Two methods are compared: one using a finite element model and the other a lumped mass-spring apparoach. In both cases a number of assumptions are made on the basis of studies in other references. The authors conclude that the results obtained by the two methods are similar. Actually, both methods appear to predict a variation of bending moment with depth that is adversely influenced by the discretization schemes. However, to judge the conclusions properly, it would be necessary to see many more details of both the methods and the results.

Model Tests

Several of the papers already reviewed have employed small scale or model tests; e.g. Papers 10/1, 10/9, 10/30 and to some extent 10/23, 10/28 and 10/32. In these papers, little or no attention has been given to the applicability of such models and to the scaling laws that might apply. In this connection, the need to consider the amplitudes of dynamic strains and the level of initial effective stress has already been mentioned.

One paper -- Paper 10/8 by *Gudehus and Hettler* deals specifically with physical modelling. The paper is in two parts, the first concerned with monotonically increasing loads and the second with shakedown during cyclic loading.

In the first part, modelling laws are postulated for two situations:

$$\text{rigid structure: } u/\ell = B(F/\gamma\ell^3)^\alpha$$

$$\text{flexible structure: } u/\ell = B(F/\gamma\ell^2\ell_e)^\alpha$$

where u = displacement, ℓ = characteristic length (e.g. width of a foundation), F = force, γ = unit weight of soil, ℓ_e = an elastic length (e.g. $\sqrt[4]{EI/\gamma\ell}$ for a pile with flexural stiffness EI) and B and α are factors to be determined from tests. These modelling laws result from a simplified stress-strain law, proposed in 1977 by Dietrich, of the form: $\Delta\varepsilon \sim (\Delta\sigma/\sigma_0)^\alpha$ where $\Delta\varepsilon$ = increment of strain, $\Delta\sigma$ = increment of stress and σ_0 = initial stress. Experiments were run using rigid model footings from 2.6 to 49 cm in radius and model piles from 9.6 to 82 cm in length, with sand as the soil. Remarkably consistent results are reported (with $\alpha \approx 1.6$ in both cases), provided that the initial stress ratios σ_1/σ_3 incremental stress ratios and stress history are the same at all scales.

For cyclic loading, the law: $u/\ell = (u/\ell)_1\, h(N)$, also proposed earlier by Dietrich, is investigated, where $(u/\ell)_1$ is the value of u/ℓ for monotonic loading and $h(N) = 1 + \beta \ln N$, N being the number of cycles. Model tests on piles yielded $\beta = 0.195$ for a range of loadings, as did an in-situ pile test. At larger cyclic loadings, incremental collapse may develop. Now the scaling laws no longer work, presumably because the size of the sand particles (which is not scaled) dictates the thickness of shear zones and hence stress changes associated with dilatancy.

The matter of appropriate scaling laws is quite important, as there appears to be increasing interest in model tests. For example, extensive model test facilities have been developed at the Public Works Research Institute (and other institutes) in Tsukuba Science City in Tokyo. Yoshimi and Tokimatsu (1978) have made excellent use of small scale experiments in their studies of two-dimensional pore pressure changes and settlement of buildings during earthquakes (see Yoshimi, 1980). This interest no doubt results from the desire to test earth structures and soil-structure systems under extreme environmental-type loadings without having to wait for nature to cause such loadings on actual structures.

A special type of model test is that carried out on a centrifuge. Centrifuge testing starts from the viewpoint that the stress-strain behavior of soil is strongly dependent upon stress level, and hence it is necessary to reproduce in the model the actual level of stress expected in a prototype. While most centrifugal testing has involved static loadings, there have been tests simulating the relatively slow cyclic loadings caused by extreme ocean waves (Rowe et al., 1976, 1977; Scott, 1979). There have also been test programs in England, the USSR and the USA studying cratering by explosions. Recently, the capability for simulating earthquake-type ground shaking has been developed at Cambridge University in England (Morris, 1979; Whitman et al., 1981). Currently there is interest in the testing of dams, and efforts to develop more versatile shaking devices have been underway by Arulandandan at the University of California at Davis.

Centrifuge tests are limited as to size, and are relatively expensive. Hence it is quite desirable to know whether and when model tests in normal gravity can give useful results. Looking more broadly, there is the question as to whether we should speak of "model tests". Perhaps they are better regarded as "small scale tests" which reproduce the main features of full scale situations and against which theoretical methods can be calibrated. These various questions all deserve much further attention.

In closing, the Reporters wish to leave you with three final observations.

First, there is extensive use of theory - often quite sophisticated theory - in connection with the soil dynamics aspects of the design of engineering projects. We applaud and support this practice, assuming of course that the results of theoretical calculations are used to supplement, and not to replace, experience and judgement. Use should also always be made of simple calculations to check the reasonableness of results from complex calculations. Use of theory seems more common for dynamic problems than for static problems. This is due in part to the small strains which occur within the soil in many dynamic problems. In those dynamic problems involving large strains, theory is still much less reliable. At present, there is a good balance between the development of theory and progress in laboratory testing. We hope this balance is maintained, and that the profession can avoid the trend - which has occurred in other disciplines - for theoretical developments to outdistance this experimental basis.

In this connection, it is essential that opportunities be both seized and created for comparing the predictions of theory against actual observations. Especially when dealing with the effects of earthquakes or extreme water waves, the collection of actual data in the field is expensive, time-consuming and subject to nature's own schedule. There have been excellent analyses of many case studies which nature has provided, and future opportunities must be pursued diligently. Class A predictions, in the purest sense, are difficult, since the exact details of the loading cannot be known in advance. It is, therefore, essential to report the results of the first attempt to use an existing theory plus initial selection of parameters, to predict observations, and not just final results showing that theory and/or input parameters can be modified to provide a good fit to recorded data.

Because case studies involving actual extreme environmental loadings occur infrequently, model tests can provide an alternative source of data against which theories and procedures for selecting parameters may be checked.

It is evident that the field of soil dynamics has continued to grow rapidly during the four years since the Conference in Tokyo. Subspecialties, such as soil dynamics for offshore engineering and geotechnical earthquake engineering, have appeared, and we find that international conferences concerning just these sub-specialties are being held. Such specialized conferences are valuable for the interchange of information at a detailed level. However, we note an increasing tendency for lack of communication between workers in different specialties. We are particularly concerned about the need to bridge between the work in soil statics and soil dynamics. Modulus, for example, usually is treated quite differently in statics and dynamics, and yet there must be continuity of stress-strain behavior with frequency.

Here lies a challenge for the International Society. The Society must on one hand help foster growth within specialties while on the other hand encouraging and aiding communication among the specialized interest groups. In particular, the Society must stimulate specialists to relate their work to the overall pattern of advance within soil mechanics and foundation engineering. These are not new problems nor new goals for the International Society, but we urge that special attention be given to this challenge during the next four years.

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