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General Report Session 1: Static and Dynamic Soil Parameters and Constitutive Relations of Soils

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Static and Dynamic Soil Parameters and Constitutive Relations of Soils

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INTRODUCTION

The soil parameters that are commonly of interest in dealing with earthquake and soil dynamics problems are:

1) The maximum shear modulus G_0, or the shear wave velocity, V_s.

2) The variation of secant shear modulus with shear strain level, G/G_0 vs γ.

3) Equivalent viscous damping as a function of strain.

4) Other parameters such as, dilatometer modulus, normalized SPT or CPT value, state parameters, formation factor.

In terms of constitutive relations the following models have been used:

1) Linear elastic (total stress) - appropriate at very small strains $< 10^{-3}\%$.

2) Equivalent linear elastic with equivalent viscous damping to account for hysteretic damping (total stress). Where pore pressure rise and liquefaction is of concern, this approach is used to obtain the dynamic stresses only. The dynamic strains and displacements are obtained from a separate procedure.

3) Incremental elastic with rules for loading and unloading (total stress).

4) Incremental elastic with shear-volume coupling effects to allow pore pressure generation on a per cycle basis for undrained conditions (loose-coupled effective stress)

5) Plastic and viscoplastic models with inherent shear-volume coupling effects to allow a fully coupled effective stress analysis.

CLASSIFICATION OF PAPERS

The 31 papers for this session are divided into two major categories: EXPERIMENTAL; (a) Laboratory, (b) In Situ; and, THEORETICAL.

EXPERIMENTAL - Laboratory (12 papers)

- Dynamic Modulus and Damping (5 papers)
- Stress-Strain, Strength and Deformation Behaviour (5 papers)
- Properties of Reinforced Soils (2 papers)

EXPERIMENTAL - In Situ (7 papers)

- Shear Modulus and Damping (5 papers)
- Shear Stress-Strain Behaviour (1 paper)
- Formation Factor (1 paper)

THEORETICAL (12 papers)

- Soil Parameters (4 papers)
- Stress-Strain Relations (8 papers)

The authors represent 13 countries: Australia (1), Canada (4); China (5); Czechoslovakia (1); Finland (1); France (3); India (1); Japan (5); Norway (1); Singapore (1); United Kingdom (1); United States (6); and, Yugoslavia (1).

EXPERIMENTAL - LABORATORY

Dynamic Modulus and Damping

Paper 1.10 by Yu & Qin presents an interesting study of dynamic properties of saturated coal fly ash in comparison with tailing sand and slime. Dynamic properties studied include shear modulus, damping, and cyclic strength. Fly ash produced by thermal power plants has its special characteristics: high fine contents, non-plastic, but pozzolanic, due to high temperature during combustion. They show that due to these characteristics:

1) The stress modulus is smaller than for sand at the same relative density.
2) The G/G_0 and D(%) vs γ relation is very similar to that for fine sand.
3) There is significant aging effect on the modulus and strength, but not significantly on the attenuation curve of stress modulus ratio and damping curves. An aging time of 180 days may increase G_0 by 75% to 400%, and cyclic strength by 100% to 500%.

The authors also compare shear modulus and damping variations for coal fly ash with curves presented by Hardin and Drnevich and Seed and Idriss. The maximum modulus, G_0 is expressed by:

$$G_0 = C\,P_a\left(\frac{\sigma_a+\sigma_p}{2P_a}\right)^{1/2} \qquad (1)$$

The modulus number C for a variety of fly ash materials is shown in Fig. 2 of their paper and compares favourably with Hardin and Drnevich for sands. The authors' data also indicates that C depends on density as prescribed by e, rather than relative density.

The authors also show a very interesting linear correlation between the cyclic resistance ratio CSR, and shear modulus number C as shown in their Fig. 14. G_0 is related to the shear wave velocity, V_s, through

$$G_0 = \rho \, V_s^2 \qquad (2)$$

Hence

$$C = \frac{\rho \, V_s^2}{P_a \left(\frac{\sigma_a + \sigma_p}{2P_a}\right)^{1/2}} \qquad (3)$$

We might therefore expect the CSR to be related to the normalized shear wave velocity $(V_s)_1$ as has been suggested by Robertson (1990), where

$$(V_s)_1 = \frac{V_s}{\left(\frac{\sigma_a + \sigma_p}{2P_a}\right)^{1/4}} \qquad (4)$$

or perhaps,

$$(V_s)_1 = \frac{V_s}{\left(\frac{\sigma_a \cdot \sigma_p}{2P_a^2}\right)^{1/8}} \qquad (5)$$

Paper 1.29 by Du, Zhu & Wu presents results of resonant column tests on both carbonate and silica sands. The sands were all tested dry at the minimum void ratio, e_{min}, which varied between 0.98 and 1.62 for the 4 carbonate sands and 0.51 to 0.57 for quartz and silica sands, LB and CF.

The maximum shear modulus, G_0, as a function of confining stress is shown in Fig. 3 of their paper. The carbonate sands have similar moduli and are significantly softer than the Leighton-Buzzard (LB) sand. The stress-strain relation for carbonate sand in the small strain region can be represented by a hyperbolic model. In addition, due to the easy breakage of cemented particles, significant change in dynamic properties with confining stress is observed.

Paper 1.40 by Teachavorasinskun et al. examines modulus reduction and damping values as a function of strain level. The results of a detailed laboratory study are presented. The results indicate:

1) That for shear strains less than $7.(10)^{-4}\%$, sands are essentially elastic and the modulus is independent of the loading type: static, dynamic or cyclic

2) The relationship

$$G/G_0 = \frac{1}{1 + \gamma/(\gamma)_{50}} \qquad (6)$$

is in good agreement with the data and was scarcely affected by the kind of sand, sample preparation, degree of saturation and confining stress. γ_{50} is the shear strain at a stress level of 50% and is similar in concept to Hardin and Drnevich's γ_{REF} (Fig. 10a).

3) The relationship between G/G_0 and damping was unaffected by confining pressure (Fig. 13c).

4) The damping of sands proposed in the past appears to be unreliably high due to inaccuracies of shear strain measurements (Fig. 14).

The measured damping of the Sengenyama sand seems very low.

Paper 1.9 by Lin & Chen extend the Idriss et al. (1978) nonlinear degradation model for cyclic response of normally consolidated clays. They propose a numerical procedure from which the degradation parameter defined in the Idriss et al. model can be evaluated at different strain levels using stress controlled test data.

The degradation parameter allows the reduced secant modulus to be computed as a function of the number of load cycles. The parameter depends on the strain level and is essentially independent of the loading conditions - stress or strained controlled (Fig. 8 of their paper). The data indicates a threshold strain below which there is essentially no degradation. For the clay tested this appears to be about 0.2%.

Paper 1.1 by Mathew Raybould describes the triaxial testing facility and data acquisition system at Nottingham University. Results of cyclic test data on coarse silt contaminated with Kaolin clay are also presented. Modulus and damping variations with shear strain amplitude are presented. The testing equipment is capable of accurate stress and strain measurements over a wide strain range. The effect of frequency is also examined and indicate higher stiffness and lower damping for the faster tests.

Stress-Strain, Strength and Deformation Behaviour

Paper 1.50 by Tatsuoka et al. describes both static and cyclic tests as well as analyses carried out to assess the seismic response of a bridge founded on dense lightly cemented gravels.

Cored samples 30 cm. in diameter were taken from depths of up to 40 m (55 m below the water). 50 monotonic and 16 cyclic undrained tests were performed. No corrections for membrane penetration was applied because the cutting action produced a smooth surface. Disturbance was not evaluated but was thought to be small and on the conservative side because: (a) the gravel was lightly cemented; and (b) the material was dense and would tend to expand and become looser due to sampling. Thus any disturbance would cause the measured response to be softer and weaker than the in situ response.

The steady state or residual strength of these dense materials was not a concern as their undrained strength would exceed their drained strength. So that neither the Castro or Seed approach to residual strength was considered.

Deformations and strains were of concern as the allowable deformations were controlled by the bridge superstructure. The cyclic testing program was designed to evaluate the likely strains. These are usually evaluated by determining the cyclic resistance of the material, i.e. the cyclic stresses ratio to trigger initial liquefaction or 5% double amplitude strain. However, the dense material showed a gradual buildup of strain with numbers of cycles, and since the magnitude of these strains were of great importance they are included on the cyclic resistance plot as shown on Fig. 14 and idealized in Fig. 19 of their paper.

Seismic displacements were then computed using a dynamic stress path approach which involves the following steps:

1) A dynamic analysis to compute the cyclic stress ratios caused by the design earthquake.
2) Assess strain potentials from Fig. 19.
3) Use strain potentials in a pseudo-static finite element analysis to compute displacements.

Displacements were found to be well below allowable values.

The fundamental aspect here is the sampling and testing of "undisturbed" samples of gravel by coring.

Paper 1.55 by Normandeau & Zimmie describes the results of cyclic simple shear tests carried out on sandy silts from the Lower San Fernando Dam. The tests were carried out at three different frequencies 0.2 Hz, 0.05 Hz and 0.025 Hz. The results shown in Fig. 8 of their paper, indicate only a minor effect of frequency on response.

The authors compare their results with Newmark's inverse relationship between displacement and frequency and find that the observed effect is very much less than indicated by Newmark. The authors rightly point out in their introduction that Newmark's relationship is not strictly applicable. Newmark was considering a rigid plastic material and was accounting for displacement due to applied forces in excess of the strength of the soil. The effects seen in the tests appear to be associated with rate dependency for applied cyclic stresses less than the strength, and hence the discrepancy with Newmark is not surprising.

In **Paper 1.18 by Matsui, Abe, & Bahr**, the rise in pore pressure associated with cyclic loading is considered to induce an overconsolidation effect. The equivalent overconsolidation ratio, OCR_{eq} from Fig. 7 is defined as:

$$OCR_{eq} = \sigma_e'/\sigma_m' \qquad (7)$$

The post-cyclic response in terms of both strength and modulus reduction can be normalized with respect to OCR_{eq} as shown in Figs. 8 and 9. It may be seen that cyclic loading has only a small reduction effect on strength but a very large reduction effect on modulus.

Paper 1.64 by Hameury & Doanh presents the results of cyclic load tests on both sand and clay samples. The samples were consolidated anisotropically with $\sigma_a > \sigma_r$ and then subjected to cyclic torsion. The stress difference $\sigma_a - \sigma_r$ was maintained at all times.

The cyclic loading caused a pore pressure rise in all samples. However, the presence of the stress difference on the dense sand prevented the pore pressure rising to equal the confining stress (initial liquefaction) as shown in Fig. 2d. This is to be expected. However, under field conditions the stress difference may not be maintained and initial liquefaction could occur. This is particularly so if the stress difference arises from a locked-in condition rather than from applied loads or ground slope.

In **Paper 1.56 by Matsuzawa & Sugimura**, we initially had some trouble with the title as it appears to be a contradiction in terms. However, the title reflects some unusual tests that were carried out.

The results of cyclic loading tests on sands for three types of tests are presented:

DCU test: in which a specimen is subjected to a constant rate of compressive strain and cyclic stress simultaneously;
DU test: in which cyclic loading without initial shear stress is applied;
DTU test: in which cyclic loading with constant initial shear stress is applied.

The DCU tests are novel and the results are shown in Fig. 4. All tests start out with no static bias, but the effect of the constant rate of compressive strain is to induce a gradually increasing static bias. The results are interesting, particularly so for the loose sample. In the test for loose sand, the cyclic loading is relatively small, so that the test resembles a monotonic undrained loading condition. The tests on the denser sands with a higher cyclic loading have very unusual behaviour that is worthy of detailed examination and reflection.

The authors' also present a model based on endocronic theory and plastic work concepts for predicting stress-strain and liquefaction response. Based on the results presented, the model is in good agreement with the measurements.

Properties of Reinforced Soils

Paper 1.47 by Puri, Das & Chae presents results on the influence of vertical reinforcing on the elastic subgrade reaction modulus. The tests were carried out on model footings with dimensions up to 0.15 m. The results are summarized in Table 1 of their paper and indicate that stiffness could be improved by a factor ranging between about 1.6 and 2.8 depending on density and type of reinforcement. Change in density alone can change the stiffness by a factor of 2 in the range D_r = 45 to 70%.

The results indicate that the reinforcing is most effective at the higher densities. This would suggest that reinforcing would be most effective where it is desired to increase the stiffness beyond that which can be achieved by densification.

Paper 1.60 by Phong M. Luong describes the energy-absorbing ability of Texsol. Texsol is "a soil-fibre composite resulting from a new technique of soil reinforcement by incorporation of continuous fibres". It is not clear from this paper how this is done in either the field or in laboratory tests. Much effort is spent on describing models for soil (critical state and stress-dilatancy) that are peripheral to the issue.

Results of triaxial tests or both unreinforced and reinforced samples under monotonic and cyclic loading are presented. The results indicate that the presence of the fibre gives the soil a cohesion (Fig. 5). A fibre content of 0.1% gives a cohesion greater than 100 kPa. It is claimed that the remedial soil is more ductile and has greater liquefaction resistance.

EXPERIMENTAL - IN SITU

Shear Modulus and Damping

Paper 1.32 by Campanella & Stewart discusses practical considerations with respect to equipment and procedures that can affect the interpretation of seismic cone results. In addition, a new procedure for determining the arrival time of the shear wave is proposed. This procedure called the "cross-correlation" method is considered to be superior to the generally used "cross-over" method. The idea here is that not just the shift at one point is used (Fig. 2) but the whole curve is used to determine the shift (Fig. 3). The authors find that the new procedure is more reliable than the cross-over method.

Paper 1.33 by Stewart & Campanella describes methods of evaluating material damping from the amplitude decay of shear waves as they progress downward. Typical amplitude decays are shown in Fig. 1 of their paper. The shear strains were computed to range between 10^{-4} and 10^{-3}%.

A problem with this approach is to adequately account for geometric (non-material) damping which is generally much larger than the material damping. This is discussed in some detail in the paper. Three approches are proposed. Two of the approaches lead to negative damping in the lower silt. Only one, the "spectral slope method" yields results that are in the expected range. The authors point out that more work is needed to validate their procedure.

Paper 1.37 by Thomann & Hyrciw indicates that both the cone tip resistance, q_c and the dilatometer modulus, E_D are not reliable index measures of the maximum shear modulus, G_0. This is illustrated in Figs. 4 and 5 of their paper. In carrying out site investigations it would seem that a more direct determination of G_0 from shear wave velocity tests is appropriate where this parameter is required.

In **Paper 1.44 by Chang et al.**, normalized equivalent shear moduli (G/G_0) and their variations with shearing strain at the Lotung seismic experimental site in Taiwan were back-calculated from recorded downhole array ground motions. Ground motion data having peak ground accelerations ranging between 0.03g and 0.21g were recorded during seven earthquakes and were used in the analysis. The time histories were available at the surface, and at depths of 6, 11 and 17 m (Fig. 3).

G/G_0 variations with strain were also obtained from laboratory tests, both resonant column and cyclic shear, and compared favourably with those computed from field response (Fig. 14). These values are similar to Seed et al. (1986) lower bound values. In addition, G_0 values obtained from in situ shear wave velocities tests were in good agreement with moduli obtained from response analyses of the field data (Fig. 10).

This type of study is extremely useful as it provides a verification of the analysis procedure commonly used in seismic response analysis. The authors claim that the damping used in the analysis was based on laboratory tests, but the data is not shown. Information on the damping used would be a very useful addition to this paper.

Paper 1.62 by V.D. Miglani describes procedures for obtaining the equivalent compliance springs from field tests on a model foundation block. The model block corresponds to the standard used in the Indian code (1.5m x 0.75m by 0.7m high). Vibration tests were carried out on the block to determine the natural frequencies of the system to vertical and horizontal (combined horizontal and rocking modes) loading. The resonant frequencies are used to compute the equivalent compliance springs, and corrections for the size of the loaded area are recommended for the prototype foundation block

$$C_M = 4\pi^2 F_{nz}^2 m/A_M \quad (8)$$

and

$$C_P = C_M (A_M/A_P)^{1/2} \quad (9)$$

(for $A_P \le 10$ m², C_p constant for $A_p > 10$ m²)

where

C_M = model spring compliance modulus

f_{nz} = the measured model natural frequency for vertical loading

m = the mass of the foundation and equipment

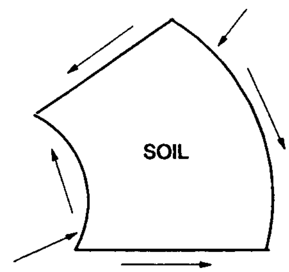
C_P = the prototype compliance modulus

A = the contact area of the foundation mass.

Field conditions can give rise to anomalies such as two resonant frequencies for vertical loading. This can arise from a non-uniform soil reaction leading to a rocking mode. These points are discussed in this paper. How valid is the area allowance factor between model and prototype tests?

<u>Shear Stress-Strain Behaviour</u>

Paper 1.48 by Henke & Henke describes a testing device to determine the in situ characteristic response of soil to seismic loading. The main elements of the device are shown in Fig. 1 of their paper, and involve testing a "hollow" cylindrical column of soil. However, unlike the usual hollow cylinder test in which both the inner and outer cylindrical faces are subjected to zero shear, in this case the shear is applied at the inner face by rotating the inner cylinder. The shear stresses applied to an element are therefore as shown.



A cyclic torque is applied to the inner active cylinder and the torque-rotation relationship observed. Both the inner and outer cylindrical faces should be smooth in the vertical direction to allow penetration, and grooved to allow development of horizontal shear on the vertical faces, an this is discussed in the paper. The authors' claim that tests can be performed in constant volume or constant pressure modes by control of the vertical piston. All the results presented are for "constant" volume test on dry sand. The reduction in stress is interpreted to be equivalent to a rise in pore pressure as is commonly assumed in constant volume simple shear tests. The presumption is made that there is zero radial displacement between inner and outer cylinders.

Observed force-rotation relationships are shown in Figs. 5 and 6 for a soil of D_r = 58% and look very similar to those observed in conventional cyclic laboratory tests. In this paper the authors don't convert torque and rotation to shear stresses and shear strains so as to compare their results quantitatively with simple shear data. This would seem to be a simple linear transformation and we wonder if this was done and how the result compared?

This is a very interesting device. Questions of sample disturbance and nonuniformity of applied stresses would spring to mind.

<u>Formation Factor</u>

Paper 1.49 by Lien Kwei Chien presents some electrical conductivity test data on three sands. Relationships between the vertical formations factor (conductivity measurement), porosity and friction angle at failure are found.

THEORETICAL

Soil Parameters

Paper 1.5 by Phoung Truong addresses the response of a mass on an elastic half-space. New values for the equivalent dynamic compliance springs and dampers for horizontal, vertical rocking, and torsional modes of vibration are presented.

The springs and dashpots appear to be linear so that no permanent displacement would result. Equations for the coupled sliding and rocking problem are presented and include Coulomb friction forces which could result in permanent displacements. However, no rules regarding the behaviour of the combined spring and friction forces are given and no examples are given to back the conclusion that the computed permanent displacements agree well with experimental results.

In **Paper 1.25 by Byrne, Salgado & Howie,** the unload-reload modulus as determined from pressuremeter tests is used as a basis to evaluate the maximum shear modulus G_{max} or G_0 in sands. An analysis is first presented to account for the varying level of shear strain within the domain as well as stress and void ratio changes. The predictions of the analysis are presented in the form of a chart (Fig. 9). The predictions are compared with both laboratory and field pressuremeter tests in which G_0 values were known from either resonant column or shear wave velocity tests.

The results indicate that G_0 can be adequately estimated from pressuremeter unload-reload tests using the proposed chart provided a correction for disturbance of about 1.4 is applied. The correction is essentially the same for self-bored and full displacement pressuremeters. An additional correction to account for anisotropy is also required for loading in the vertical plane as opposed to the horizontal plane. This factor is about 1.2.

The results suggest that if moduli alone are desired, shear wave velocity measurements would be a more direct approach.

Paper 1.34 by Misra & Chang presents a particulate approach for modelling the response of cemented sands under small strain condition. The derived mathematical model is based on classical Hertzian contact theory. It includes adhesion forces at the particle contacts and accounts for particle grain properties (e.g. particle stiffness), particle contact properties (e.g. contact adhesion and friction) and particle packing properties (e.g. void ratio). The approach brings physical insights into the small strain response of cemented sands from a micro-mechanical point of view. Application of such a model would call for an understanding of soil response and detailed property measurements at the particle level, as advocated by Scott (1987).

Paper 1.8 by Svoboda discusses concepts of soil behaviour and response to dynamic loading. Porosity and vibration velocity are considered to be key factors.

Stress-Strain Relations

Paper 1.57 by Kvasnicka & Ivsic proposes a new method of predicting pore pressure rise in sand under undrained cyclic loading. The method relates the residual pore pressure rise in sand to the normalized shear work (Moroto's parameter) during cyclic loading and the state parameter of the soil. Test data are presented to show the cyclic pore pressure rise as a function of Moroto's parameter, S_m, for a range of relative densities or state parameters. These are then normalized with respect to state parameter and form a unique curve (Fig. 2). This looks promising. We would expect that the shear work should be the plastic or hysteretic work and this could be important at low levels of applied shear strain where a threshold value likely exists below which pore pressures do not develop.

Paper 1.24 by Byrne presents a simple two parameter model for predicting the plastic volumetric strains induced by cyclic loading. These plastic volume strains in turn are used to predict pore pressures for undrained cyclic loading conditions. The model is both a simplification and an extension of the Martin et al. (1976) model and is useful for loose-coupled effective stress dynamic analyses. The model is calibrated against both laboratory and field data and its predictions of volumetric strain, pore pressure rise and liquefaction assessment are shown to be in good agreement with the observations (Figs. 4, 7, 8, 9, and 11). The concept of a threshold shear, γ_t, strain was found to be important. The data examined suggests that γ_t is in the range .002% to .01%. For $\gamma < \gamma_t$, no excess pore pressures are generated.

Paper 1.12 by Chung-Jung Lee is an interesting paper in which the stress-dilatancy equation is used to model cyclic simple shear conditions and the results compared with constant normal stress and constant volume cyclic test data. The basic dilatancy equation is:

$$\pm\tau/\sigma_n = \pm \tan\phi_\mu \pm \alpha(\theta)\,\frac{d\epsilon}{d\gamma} \qquad (10)$$

in which

- τ = the shear stress on the slip plane (plane of maximum obliquity)
- σ_n = the normal stress
- ϕ_μ = basic friction angle
- $\alpha(\theta)$ = a positive constant
- $d\epsilon$ and $d\gamma$ = the normal and shear strains on the slip plane

The author shows this equation to be in good agreement with test data for Ottawa and Fulong sands as shown in Figs. 1 and 2 of his paper.

For the constant volume test he argues that the same plastic slip will occur and be balanced by elastic rebound. Therefore $d\epsilon = d\epsilon^e + d\epsilon^p = 0$ and $d\epsilon^e = -\Delta\sigma'/M$. Hence

$$\pm\tau/\sigma' = \pm \tau\tan\phi_\mu \mp \alpha(\theta)/M\; d\sigma'/d\gamma \qquad (11)$$

Again the test results shown in Figs. 3 and 4 of the paper are in good agreement with the model results.

This is a simple model that looks promising for predicting pore pressure fluctuations during cycles of load. However, it does not appear to address the problem of pore pressure increase from cycle to cycle.

Paper 1.14 by Ronaldo I. Borja presents a conceptual framework for capturing the rate dependency of soil based on viscoplastic theory. Examples of predicted stress-strain response for monotonic and cyclic loading are presented, and the results are interesting. Could shear-volume coupling effects be included to allow pore pressure to be predicted?

Paper 1.52 by Doahn extended his nonlinear incremental model to cyclic loading condition and compares the model prediction with test results from a series of drained two-way triaxial strain-controlled tests on sand. His model predictions for plastic volume change with number of cycles of axial strain appear to be in very good agreement with the measurements (Figs. 3 and 4). However, in this paper, there is very little detail given on the model itself.

In **Paper 1.3 by Gutierrez, Ishihara & Towata**, a sophisticated plastic stress-strain model is proposed for sand. The directions of the strain increment depends on both the stress state as well as the direction of the stress increment as shown in Fig. 3. Only a very small elastic area is considered so that plastic deformations occur for all load increment directions. The plastic hardening modulus is a product of two functions: H_1, reflecting shear stress level; and H_2, reflecting the stiffening effect of the accumulated normalized plastic work. Predicted and observed strain increments during cycles of principal stress rotations are shown to be in good agreement (Fig. 5).

Paper 1.31 by Kaliakin presents an elastoplastic-viscoplastic model based on bounding surface concepts, and examines its capability in predicting response of cohesive soils subjected to cyclic loading by comparison with experimental data. The salient feature of this approach is its ability to model the inelastic strains that occur when the stress state lies within or on the bounding surface. The magnitude of this inelastic strain depends upon the distance between the stress point and the stress point "image" on the bounding surface, and two different hardening criteria are used for plastic strains within and on the bounding surface. Associated flow rule and isotropic hardening condition are employed. The formulation has great flexibility in capturing general soil response under both drained and undrained conditions. However, further calibration is needed to obtain better agreement with experimental data.

Paper 1.46 by Selnes & Nodim presents an interesting parametric study on the effects of soil stress-strain hysteretic shapes on dynamic response (Fig. 2). Their study involves comparing the classical solutions for the visco-elastic or hysteretic material using a secant modulus and equivalent damping from the material hysteretic loop with that using the direct integration of the nonlinear equation of motion. The study shows that although the area of the hysteresis loop provides a good measure of the damping for highly nonlinear, irregular hysteresis soil behaviour, the irregular hysteresis that is often observed in cyclic soil testing may cause a significant shift in resonant frequencies as compared to the classical solutions (Fig. 5). The highly nonlinear and irregular hysteresis loop may produce high amplification not only at the resonant frequency but also at other higher frequencies (Fig. 6). These results deserve further attention, especially for massive structures undergoing nonlinear soil-structure interaction.

SUMMARY AND CONCLUSIONS

From the papers presented in this session, the following summary and conclusions may be made:

Experimental

- The in situ G_0 is best obtained from in situ shear wave velocity tests.
- Much laboratory data on G/G_0 variations with shear strain are presented. It would appear that for sands there is a near unique relationship between G/G_0 and normalized strain γ/γ_{REF} or γ/γ_{50} where $\gamma_{REF} = \tau_f/G_0$ and γ_{50} is the strain at $\tau/\tau_f = 50\%$.
- Much laboratory data on equivalent viscous damping, D, as a function of strain are presented. Damping is hysteretic and related to the G/G_0.

$$D = D_{max}\,(1 - G/G_0) + D_{min} \qquad (12)$$

where D_{min} is of the order of 1 to 2%.

- There is a threshold shear strain below which no plastic volumetric strain or pore pressure generation occurs in undrained tests. This is in the range 2×10^{-3} to 10^{-2}% for sands.
- Undrained cyclic loading of saturated silt and clays causes a pore pressure rise which reduces its post cyclic stiffness and strength. The pore pressure rise effect of cyclic loading can be simulated by testing an overconsolidated sample at the same equivalent overconsolidation ratio. The cyclic loading or overconsolidation of silts and clays causes a major reduction in stiffness but only a minor reduction in strength. Could the post-cyclic stress-strain response of sand also be simulated by overconsolidation?
- Reinforcing can increase the modulus by a factor of about 2. The effect is most pronounced on dense soils. This suggests that where higher foundation moduli are required, densification should first be considered. Where this is not adequate, reinforcing can be considered.

Theoretical

- The parameters for use in equivalent elastic analyses have not significantly changed in the past 20 years.
- Minor improvements in shear induced plastic volumetric strains and pore pressure generation are suggested for the loose-coupled procedure.
- A simple stress-dilatancy type model as presented in Paper 1.12 could be very useful for predicting response when the phase transformation state is reached.
- The strains prior to reaching the phase transformation state in sands are generally small. Thereafter they may be very large, particularly if the soil is loose. There is a great need to evaluate strains for undrained loading along the phase transformation line.
- Plasticity models that incorporate shear-volume coupling effects are very complex. The papers in this area are very interesting and need further verification with laboratory data.

TOPICS FOR DISCUSSION

Experimental

1. The maximum shear modulus G_0 is an important parameter. It depends on particle shape and structure, void ratio and stress level and can be expressed as:

$$G_0 = A \cdot F(e)\; P_a(\sigma^*)^n \qquad (13)$$

where

A = a factor that depends on particle shape and structure

$F(e)$ = a void ratio function

P_a = atmospheric pressure

σ^* = a normalized stress that could be:

$\frac{\sigma'_m}{P_a}$ - The mean stress

$\frac{\sigma'_a+\sigma'_p}{2P_a}$ - The average stress

$\frac{\sigma'_a \cdot \sigma'_p}{P_a^2}$ - The individual stress

where σ'_a and σ'_p are the effective stresses in the "loaded plane".

In addition, it appears that G_0 depends on stress ratio. The product $A \cdot F(e)$ can be taken as a normalized modulus and used not only for obtaining modulus under changed stress conditions but as an index of behaviour, such as a measure of liquefaction resistance (Paper 1.10). The appropriate stress function and the effects of stress ratio are therefore important factors to consider.

2. Are G/G_0 unique functions of normalized strain γ/γ_{REF} for most soils? Do we need to obtain them from testing at each important site?

3. Can the appropriate subgrade reaction or compliance modulus for foundation vibration problems be adequately specified from G_0 and G/G_0 considerations?

4. Can material damping be adequately specified in the form:

$$D = D_{max} (1 - G/G_0) + D_{min} \qquad (14)$$

Do we have enough information on D_{max} and D_{min} for the various soil types?

5. Is there a need to concentrate on in situ determinations of modulus reduction and damping parameters?

Analytical

The type of constitutive relation required depends on the level of dynamic analysis considered appropriate.

1. Is the commonly used equivalent viscoelastic analysis adequate:

   a) when the strains are small and plastic volumetric strains and pore pressure rise do not occur? The computed stresses, strains and displacement are generally considered reliable in this case.

   b) when pore pressure rise and liquefaction would occur. In this case only the dynamic stresses would be accepted, and are used in a secondary procedure to assess the triggering of liquefaction and/or strains due to cyclic mobility?

2. Are loose-coupled incremental elastic analyses with pore pressure effects included on a per cycle basis adequate effective stress analyses? If so, are they necessary and when are they necessary?

3. The required stress-strain relations for a coupled effective stress analysis are very complex. More calibration with laboratory and field measurements are required here.

REFERENCES

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