04 Apr 1995, 10:35 am - 10:55 am

General Report —Session I: Static and Dynamic Engineering Soil Parameters and Constitutive Relations of Soils

Peter M. Byrne
University of British Columbia, Canada

Li Yan
Klohn-Crippen Consultants Ltd., Canada

Proceedings: Third International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics
April 2-7, 1995; Volume III, St. Louis, Missouri

General Report - Session I
Static and Dynamic Engineering Soil Parameters and Constitutive Relations of Soils

Peter M. Byrne
University of British Columbia
Vancouver, B.C., Canada

Li Yan
Klohn-Crippen Consultants Ltd.
Richmond, B.D., Canada

INTRODUCTION

A total of twenty-seven papers were received for this session. The papers can be broadly classified as Experimental Work and Analytical Methods and Applications.

Experimental (mainly laboratory work) (21 papers)

- Stiffness and Damping (10)
- Stress-strain, Strength and Deformation Behaviour (9)
- Soil-structure Interface Properties (2)

Analytical Methods and Applications (6 papers)

- Cyclic Stress-strain Behaviour, Shear Strength (3)
- Dynamic Analysis of Soil Response and Soil-structure Interaction (2)
- Risk and Reliability (1)

The authors of these papers represent 10 countries: Australia, Canada, Chile, China, Italy, Iran, Japan, Spain, U.K., U.S.A.

EXPERIMENTAL WORK

Stiffness and Damping

Paper #1.06 by Wang et al. presented shear modulus (G) and damping ratio (D) results obtained from resonant column tests and cyclic triaxial tests carried out in the high pressure range (from 150 psi to 500 psi). The soils tested included Ottawa sand and natural soil samples (silty sand to clayey sand, and clay) taken from thin-walled tube samples from a 1200 ft deep drill hole. The G and D values obtained under different strain levels were compared with the Hardin and Drnevich (1972) model and Seed (1984) curves.

They found that for reconstituted Ottawa sand samples the Hardin and Drnevich model fitted well with their data (both G and D values) although the model was originally developed for confining pressures less than 100 psi. The G values of Ottawa sand obtained at high pressure were found to lie above the range defined by Seed's curves for sands, whereas the D values were generally found to fall within Seed's range but close to the lower bound.

For natural soil samples, the G data were predicted quite well by the Hardin-Drnevich model for both silty to clayey sand and clay samples, however, the measured D values were higher in the low strain range than those predicted by the Hardin-Drnevich model.

No frequency effects were found on either shear modulus and damping ratio at the high pressure range. This is consistent with findings in the low pressure range.

Paper #1.22 by Nakagwa and Soga describe a cyclic torsional shear testing system to measure dynamic properties of soils over a wide range of strain (10^{-4} % to 1%). A proximity transducer was used to measure very small displacement in the range of resonant column tests. Soils tested included sands, and undisturbed and reconstituted clay samples. Results from the new cyclic torsional shearing test device showed that the shear modulus degradation curves for materials tested can be better fitted by using a new two parameter model:

$$\frac{G}{G_{max}} = \frac{1}{1 + \alpha \gamma^{\beta}} \qquad (1)$$

where G_{max} is the maximum shear modulus at γ=<10^{-4} %, and α and β are material constants. Their test results also showed that an empirical relation appears to exist between parameters α and β which is independent of the material tested.

Paper #1.52 by Rodriguez-Roa and Palma described a laboratory study carried out to examine various factors that affect the dynamic stiffness and damping ratio of a granular soil from Santiago, Chile. Factors examined included grain size distribution, degree of compaction, confining pressure, magnitude of the cyclic stress and number of loading cycles. Cyclic triaxial test equipment was used.

Results showed that apart from the strain level effects, the confining pressure and degree of compaction were the most significant factors affecting the magnitude of the dynamic shear modulus. The grain size distribution is a secondary factor and the number of loading cycles is the least important factor. For the damping ratio, however, due to the scatter of the experimental results, the relative order of influence from the factors studied is not as clear. The banana shaped stress-strain curves obtained from the testing do not lend themselves to an equivalent viscoelastic interpretation.

Paper #1.32 by Lo Presti et al. used the conventional triaxial compression loading to study the stress dependence of sand stiffness. The tests were carried out over a wide range of strain from very small to very large (up to 2%). Two types of sands were used: uniform fine quartz sand with no fines, and a well graded coarse to medium carbonatic, crushable sand with about 2% fines. Tests were carried out on specimens consolidated to either constant vertical stress or constant horizontal stress with a range of consolidation stress ratio, $\sigma'_{hc}/\sigma'_{vc}$. Their results showed that the secant Young's modulus at small strain is only dependent on the vertical (axial) effective stress and not on the horizontal stress. On the other hand, the secant Young's modulus at larger strains seems to be controlled by the horizontal effective stress. The Young's modulus of the quartz sand is independent of the loading history, while the Young's modulus of the calcareous sand increases with the over-consolidation ratio due to its crushability. Comparison of Young's modulus from triaxial compression tests and the shear modulus from the torsional shear tests on the same sands showed that these two moduli can be related through isotropic elasticity theory at low strain but not at large strain. This indicates that the small strain elastic response is isotropic while the large strain plastic response is anisotropic.

Paper #1.35 by Nazarian and Baig presented an evaluation of the use of Bender Element in measurement of dynamic shear modulus in coarse grained material, i.e. coarse grained sand and glass beads, for a range of relative densities. The shear moduli measured from bender elements in a triaxial test device were compared with those from resonant column tests. Results from the bender element showed good repeatability for each relative density. Good comparison was obtained between results from bender element and resonant column tests. Shear moduli from the bender element under different stress level were compared with some empirical correlations. Good agreement was found for both low to high relative density. This study indicated that the bender element can work well with a large range of particle sizes and shapes.

Paper #1.43 by Zhang and Aggour compared the results of the maximum shear modulus, G_{max}, from resonant column tests under different types of loading. The loading types examined included sinusoidal, random and impulsive loads. Ottawa 20-30 sand was used in the study. They found that different loading types have no apparent effects on the G_{max} of sands, and the G_{max} under different loadings compared well with the published equations, such as Hardin (1978).

Paper #1.30 by Chaney compared the moduli obtained from uniform cyclic stress and those from random irregular loading history for both undisturbed and remolded silty sand samples. He also studied the effects of stress history on dynamic shear modulus. Tests were carried out in the cyclic triaxial test device. The random loading time history comprised of a portion of the 1940 El Centro S90W earthquake record. Shear modulus and damping ratio were evaluated using hysteresis loops.

Results showed that at the low shear strain level, shear modulus and damping ratio computed from three different cycle conditions (before peak, at the peak, and after peak) from the three cycle conditions are similar, however, at the high strain level, the "before peak" cycle tends to give higher modulus and lower damping for the dense sample. Comparison of shear modulus and damping from uniform and random loading cycles showed at the low strain levels ($<10^{-1}$ %), the modulus and damping from both types of loading are very similar while at higher strains ($>10^{-1}$ %) the moduli from random loading cycle are higher. Results seemed to suggest that random loading would give higher damping ratio.

Paper #1.19 by Zaman et al. presented cyclic testing of aggregates for pavement design. Resilient modulus was evaluated using repeated (cyclic) triaxial tests according to guidelines stipulated in both AASHTO T292-91I and T294-92I. Results showed that the T294-92I testing procedure gives higher resilient moduli than those obtained by the T292-91I testing procedure. Following the repeated triaxial testing static triaxial compression tests were also performed to evaluate the static strength of the material. These test showed that cohesion increases while friction decreases because of the conditioning induced by the dynamic stress. They also showed it is possible to establish a correlation between the resilient modulus and the cohesion and friction angle of the aggregates.

Paper #1.13 by Ashmawy et al. presented a comprehensive review of various definitions of damping used in dynamic analysis. The relationships and implication of different types of damping for dynamic geotechnical analysis were given. A state of practice in measurement of soil damping was also reviewed in terms of laboratory and in-situ measurements. For the laboratory evaluation of damping, various factors such as type of apparatus, loading path, rate of loading, and strain level, and sample disturbance were discussed and contrasted with different in-situ damping measurement techniques. Preliminary laboratory data on a kaolinite specimen showed sample disturbance has significant effect at large strain, reducing damping ratio at large strain. The paper discussed different unresolved issues related to in-situ damping measurements. They stressed a need for a model or methods to separate geometry (radiation) damping in reduction of downhole and crosshole data for soil damping measurements. The paper also emphasized the importance of measuring damping values in the vicinity of the resonant frequency of the specific dynamic problem in question.

Paper #1.07 by Cuellar et al. described the geotechnical field investigations conducted to characterize the static and dynamic properties of the foundation soil and structural material of the Comares tower in the Alhambra palace, Granada, Spain. The field investigations were carried out for evaluation of the seismic risk as this tower was built in the early thirteenth century, and located in one of the most seismically active region in Spain.

The foundation soil for the tower consists mainly of a conglomerate with 60% quartzite and schist particles of gravel size embedded in a silty clay matrix mass. The geotechnical properties of the foundation soil were determined by in-situ horizontal plate loading tests and surface wave propagation tests carried out in two 2 m deep trenches excavated near the tower. Surface wave tests were used to estimate shear wave velocity and hence the maximum shear modulus profile in the ground. The shear modulus and damping ratio versus strain level relationships were assumed to follow a hyperbolic relationship and were estimated from the cyclic plate load test results. The secant modulus from the plate load test was found to range between 0.15 and 0.25 of the maximum modulus from the wave propagation tests.

Stress-strain, Strength and Deformation Behaviour

Paper #1.09 by Chang and Hwang presented a useful case study which compared the cyclic shear strengths of soils obtained from different methods for a recent alluvial site in Taiwan. The subsurface layers of soil at this site comprise of recent alluvial deposits of loose silty sand and sandy silts. An intensive in-situ testing program, including the standard penetration test (SPT) with energy measurements, the seismic cone penetration test (SCPT), the cross-hole Vs measurement, the dilatometer test (DMT), and the pore water pressure measurement were carefully conducted at the site. Undisturbed tube and block samples were also taken for laboratory strength testing in a 2 m diameter hand-dug vertical shaft. All samples were free drained and frozen at the site and then transported to the laboratory.

The cyclic shear strengths of block and tube samples from the shaft were tested in the triaxial apparatus. The field strengths $(SR_{15})_f$ were estimated from the laboratory strengths using:

$$(SR_{15})_f = (SR_{15})_{triaxial} \cdot 0.9 \cdot \frac{1+2Ko}{3} \quad (2)$$

in which

$(SR_{15})_{triaxial}$ = cyclic strength after 15 cycles in cyclic triaxial test
0.9 = correction factor for two direction shaking
$(1+2Ko)/3$ = correction factor for different stress condition between the test and the field
Ko = at rest earth pressure coefficient estimated from DMT

Comparison of cyclic strengths predicted by all these methods was shown in Fig. 10 of their paper. The laboratory strengths from block samples are significantly higher than the others. Generally, the cyclic strengths obtained by the various methods exhibit the following trend: block sample > tube sample ≈ SPT-N method > Vs method > CPT-q_c method.

The undrained strength of soil is generally highly dependent on the direction of loading as compared to the direction of deposition. Vertical loading as occurs in the conventional triaxial test will generally give much higher undrained strengths than simple shear or extension testing and could account for the higher strengths from laboratory test data as opposed to field SPT based field experience.

Paper #1.23 by Pradhan et al. discussed the mechanism of migration of excess pore pressure in a clay sample and subsequent consolidation behaviour of clay after the dissipation of excess pore pressure following an undrained cyclic loading. A simple technique for measuring pore pressure at the mid-height periphery of the specimen is proposed. It consists of a tube which penetrates the membrane at the mid-height and connects to a filter drain while the other end connects to a transducer placed outside the cell. By comparing the pore pressure measurements at both ends and at the mid-height of the specimen, the pore pressure distribution along the specimen height was clearly shown to be non-uniform and dependent on the cyclic loading frequency. It was shown that the peak pore pressures at the sample ends were always higher than that at the mid-height irrespective of frequency and stress level until a certain level of pore pressure ratio was reached. At higher pore pressure ratios which are associated with large strain, the peak pore pressure at the mid-height becomes larger than that at the sample ends. Depending upon the excess pore pressure ratio and soil permeability, significant time may be required for pore pressure to migrate and achieve an equilibrium condition within the sample. The use of frictionless and platens could alleviate this problem for cyclic triaxial tests.

Subsequent consolidation after undrained cyclic loading showed that the recompression index decreases slightly at the beginning of the consolidation and increases at the final stage of consolidation. The recompression index from cyclic triaxial tests are also compared with those from torsional simple shear tests. Good agreement was found when the index from torsional simple shear tests was computed using effective radial consolidation stress.

Paper #1.34 by Yang studied the effective stress paths of clayey soils under triaxial single level and multilevel repeated loadings. The studies aimed at modelling soils subjected to staged repeated loading such as from offshore wave loading conditions. The studies showed that under a single event comprising of a repeated load sequence, the pore pressure will increase with the loading cycles, resulting in an accumulation of deformation. The stress paths for soils consolidated at different consolidation pressure have a similar pattern and can be normalized by the consolidation pressure. The stress paths under multilevel repeated stress loading indicate that the preshearing and subsequent consolidation have a great effect on the effective stress path. Preshearing gives clayey soils an apparent over-consolidation effect. Low preshearing appears to inhibit the development of pore pressure and deformation during the subsequent repeated loading, provided the subsequent repeated loading does not exceed the critical repeated stress level.

Paper #1.39 by Tanaka and Shirakawa used the acoustic emission, AE, of soil to study the yield locus changes of sand during undrained cyclic loading in the triaxial device. An AE sensor was installed in the lower pedestal of the specimen to detect minute sounds which were emitted from sand particles sliding against each other when soil starts to yield with irrecoverable deformations. Results showed that the AE measurement can detect the yield point. During the undrained cyclic loading, the yield locus of sand constantly change its size and shape.

Paper #1.18 by Zhang and Shamoto conducted cyclic torsional and triaxial tests on saturated sand under Ko conditions to examine the lateral total stress change during and after undrained cyclic shearing under Ko conditions. The lateral stress change during Ko undrained shearing was studied using hollow cylindrical samples under torsional loading whereas the lateral stress change during subsequent 1-D pore pressure dissipation after undrained shearing was simulated using a triaxial test device. Results showed that during Ko undrained shearing, the total lateral stress increases almost linearly with excess pore pressure before reaching an ultimate value which is equal to the total vertical stress. During this process, the Ko value also increases almost linearly with excess pore pressure ratio before it reaches its ultimate value of unity. In addition, triaxial test data indicated that during the subsequent 1-D compression, expansion and recompression due to dissipation and redistribution of excess pore pressure, the total lateral stress also decreases and increases linearly with the excess pore pressure. However, the Ko value in terms of effective stresses changes nonlinearly with the excess pore pressure.

Paper #1.54 by Lee presented a series of triaxial test results on Likan sand to show the static shear effects on the liquefaction resistance of sand. Test results were interpreted in the framework of critical state soil mechanics and the state parameter. Two series of triaxial test were performed: (1) monotonic strained controlled drained and undrained triaxial tests for determination of steady state lines; (2) stress controlled cyclic triaxial tests on anisotropically consolidated samples with different combinations of static shear, and state parameter, Ψ.

His results showed that for Likan sand, the critical state line and the steady state line are the same, but different steady state lines are obtained for compression and extension loading paths. This would indicate that for a given initial state the state parameter depends on the direction of loading. Results showed that under monotonic undrained loading, loose sand with high Ψ experienced strain softening when its stress path in the p'-q plot hit the critical stress ratio line (CSR). For the denser samples with negative Ψ, there was no peak shear stress and the CSR line did not exist. For a given void ratio, there exists a minimum confining pressure for strain softening to be induced during an undrained shear loading. The higher the density of sands, the higher the minimum confining pressure. A sample consolidated to less than the minimum confining pressure will dilate when its stress path hits the so called "phase transformation" line or constant volume friction angle. Therefore, for a given density sand, the CSR line does not originate from the origin of the p'-q plot but from a point at the phase transformation line where p' is large enough for this density to suppress the dilation. The phase transformation line, on the other hand, originates from the origin and connect to the starting point of the CSR line. Therefore, the cyclic liquefaction resistance of a sand may decrease with increasing static shear for the contractive soil (Ψ>0), and increase with increasing a values for the dilative soils (Ψ<0), and may remain the same for sand having $\Psi \approx 0$. These observations are similar to those made by Vaid and Chern (1985) and Sladen (1987).

Paper #1.28 by Parathiras studied the residual strength of soil under fast loading rates using a ring shear apparatus. He concluded that their behaviour under fast shearing depends not only on their plasticity and the magnitude of their slow drained residual strength but also on the presence of water in their environment and matrix.

Paper #1.11 by Veyera and Ross presented a study of the stress-strain behaviour of unsaturated sands tested under a high strain rate using a Split-Hopkinson Pressure Bar (SHPB). The compacted soil specimens were subjected to undrained uniaxial confined compression at approximate strain rates of 1000/s to 2000/s. The observed stress-strain curves suggested that for the unsaturated soil, the soil skeleton dominates the response from the initial loading to a strain where soils become fully saturated. This strain level is called 'lock up strain'. For soils with low saturation, this lock-up-strain did not occur even at large strains. Results suggested that the soil behaviour is governed by the fluid phase after the lock-up-strain. The initiation of lock-up-strain is dependent on initial saturation and also varies somewhat with soil types.

Soil-structure Interface Properties

Paper #1.05 by Fakharian and Evgin presented an experimental study of soil-structure interfaces under both static and cyclic loadings. Experiments were conducted on sand and an aluminum oxide clothed steel surface using both direct shear and simple shear devices. A comparison of test results from both testing methods indicated that no major difference exists between the two types of testing in terms of peak and residual strengths. However, the simple shear device is able to provide separate information on the shearing behaviour of soil and the sliding behaviour at the soil-structure interface. The results also showed that under the displacement controlled cyclic test with a displacement amplitude less than that required to fail the interface in a monotonic shearing, the shear stress will reach the peak strength and stabilize at the post peak strength after certain number of loading cycles.

Paper #1.04 by Evgin and Fakharian presented a very interesting study of 3-dimensional cyclic behaviour of soil-structure interface. The 3-D behaviour was studied using a two directional simple shear device. During the test, a constant shear stress was applied in one direction while a cyclic shear stress was applied in the other orthogonal direction. The cyclic shear stress was applied in a displacement controlled mode.

Their experimental results indicated that the existence of a constant shear stress significantly influences the displacement-controlled cyclic behaviour of an interface in the orthogonal direction. The number of cycles required to bring the interface to failure and the magnitude of peak and residual strengths in one direction are reduced by increasing the constant shear stress in the other orthogonal direction although the resultant peak and residual strength of the interface appears to remain the same as in the absence of shear stress in the other direction. The cyclic loading in one direction induces continuous sliding in the other direction even though the shear stress in this direction remains constant. One practical implications of these results is in the case of piles subjected to both axial and lateral loads. The cyclic behaviour of laterally loaded piles may not only be a function of lateral loading conditions but could also be a function of the magnitude of axial loads carried by the piles, or *vice versa*.

ANALYTICAL METHODS AND APPLICATIONS

Cyclic Stress-strain Behaviour, Shear Strength

Paper #1.33 by Yang and Shackel presented an interesting theoretical approach for determining four of the five anisotropic elastic parameters for an elastic cross-anisotropic soil model. The four elastic parameters are solved from basic stress-strain equations for elastic cross-anisotropic material by minimizing the errors between the predicted strains and the measured strains for a given stress increment by using a least square method and two sets of stress and strain measurement data from a true triaxial test. The conditions for unique solution of the four parameters were determined, which could be satisfied by imposing certain conditions between the stress and strain increments in the true triaxial tests.

Paper #1.31 by Erten and Maher used Nemat-Nasser and Shokooh (1979) model to predict excess pore pressure built-up under cyclic triaxial loading conditions. Results showed that good agreement between experimental data and model prediction were obtained for both loose and medium sands and medium silty sand.

Paper #1.17 by Hicher and Kordjani presented a model to capture the cyclic behaviour of sand in the large strain range. They used an elastic-plastic formulation with a kinematic harding rule, and showed that it is possible to capture the cyclic behaviour of soils over a wide range of strains (10^{-6} to 10^{-1}) using their model with a set of soil parameters.

Dynamic Analysis of Soil Response and Soil-structure Interaction

Paper #1.26 by Tehranizadeh presented a parametric study of effects of soil and structure properties on elastic foundation settlement under an earthquake loading. A seven storey building resting on a mat foundation was modelled using a finite element method. A direct method was taken in which structure, foundation and surrounding soil were simulated in the finite element analysis. Soils appeared to be modelled as an elastic medium. The acceleration time history of the 1977 Tabas earthquake was applied to the system and the induced settlements of the foundation were calculated. The parameters studied were soil modulus, foundation thickness, weight of structure and number of floors. Results showed that while increasing soil modulus and foundation thickness result in smaller elastic settlements, increasing weight of the structure and number of floors lead to larger settlements. Results also showed higher differential settlements occur when higher total elastic settlements are predicted.

Paper #1.51 by Anandarajah presented a verification of an effective stress-based finite element procedure for analysis of soil and soil-structure response to earthquake loading. In this procedure, an elasto-plastic stress-strain model based on bounding surface plasticity theory and an associated flow rule was used to model the behaviour of granular material under both monotonic and cyclic loading conditions. A fully-coupled finite element formulation of Biot's equation for two-phase porous media was employed to model the pore pressure build-up and dissipation simultaneously. Verification of this procedure was carried out by comparing the prediction results with centrifuge test data. Two centrifuge test problems were analyzed (1) a soil liquefaction problem which represents a prototype of a 10 m deep saturated sand deposit subjected to a simulated earthquake motion at the bottom, (2) a soil-structure interaction problem which represents a prototype of a two-dimensional model of the Cypress Freeway that collapsed during the 1989 Loma Prieta earthquake. The model comprised of a two storey superstructure supported by two columns, each resting on a pile cap supported by 20 piles and saturated silty clay. Comparisons were made on horizontal accelerations, excess pore pressures, and ground settlement for the ground liquefaction problem, and on bending moment, horizonal displacement and acceleration in the structure members for the soil-structure interaction problem. Results are encouraging, and showed usefulness of this type of verification on a sophisticated analysis procedure in geotechnical earthquake engineering.

Paper #1.25 by Singh and Das discussed the reliability aspects in soil dynamic engineering. The authors gave a brief overview of current approaches to reliability assessment including Fuzzy set theory, Analytical and Monte Carlo simulation. Monte Carlo simulation was preferred by the authors because of its ability to handle complex problems without resorting to distortion of the actual problem and/or input probability distributions. An illustrative example using a Monte Carlo simulation of a concrete foundation subjected to a vibratory force is given in their paper.

GENERAL COMMENTS

To rationally design soil-structures, we need to first predict their response to static and dynamic loading. To do this, we must first capture the element behaviour. Once captured, we can simulate the response of a collection of such elements to either static or dynamic loading using a numerical procedure such as the finite element method. This session is generally concerned with stress-strain modelling of soil for use in such analyses.

Laboratory testing to evaluate the element behaviour is generally carried out using triaxial, simple shear, torsional and resonant column devices. For dynamic loading problems, the element stress-strain response is generally obtained from constant amplitude stress or strain tests and the results presented in the form of modulus reduction and damping curves as a function of strain level. This corresponds to an equivalent viscoelastic model and the results in this form are used in standard practice-total stress dynamic codes such as SHAKE and FLUSH which use this model.

This total stress approach is adequate when the shear strains are less than about 10^{-2} %. For shear strains larger than this, shear induced volumetric compaction occurs due to slip at grain contacts, and it is this plastic volumetric strain that causes porewater pressure rise and liquefaction, when the presence of an incompressible fluid such as water in the pores restricts drainage.

The conventional approach to the liquefaction assessment problem is to assume that the dynamic shear stresses computed from the total stress analysis are correct and to compare these stresses (appropriately modified) with the cyclic resistance of the soil. The cyclic resistance in turn is obtained either directly from cyclic triaxial or simple shear tests or indirectly from index tests such as penetration resistance together with field experience.

The zones of liquefaction are then assigned a residual strength and the soil-structure analyzed using conventional limit equilibrium analysis to assess the possibility of a flow slide. If the system is stable against a flow slide, then further analyses are required to assess the likely deformations.

The test data show that granular soils are basically linear elastic up to strains of about 10^{-3} %. For shear strains less than this, the tangent shear stiffness drops, but no plastic slip at grain contacts appears to occur until the shear strain completed exceeds about 10^{-2} %. For cyclic shear strain in excess of 10^{-1} %, very significant compaction due to slip at grain contacts occurs and leads to a large drop in effective stress if the volume is constrained either by the presence of water or if the boundaries of the element are constrained to prevent volume change from occurring. Most of experimental work on dynamic stress-strain relation to date has been presented in the form of modulus reduction and damping curves rather then geared towards the development of more fundamental incremental stress-strain modelling required for effective stress modelling of soil response.

The fundamental approach to liquefaction assessment is the effective stress method which requires an elastic-plastic skeleton stress-strain law capable of capturing the drained response from monotonic as well as cyclic loading element test conditions. The saturated undrained or partially drained response can then be predicted by imposing the volumetric constraint from the water and the results compared with element test data. In this way the element behaviour drained or undrained is first captured.

The fully coupled response of a soil structure can then be predicted using the calibrated element response and taking account of redistribution and drainage effects. The procedure should first be verified with model soil structures under controlled conditions such as centrifuge tests. The procedure could then be used with some confidence for analysis of new or existing structures, and such analysis could be very helpful in the retrofit design of existing structures.

SUMMARY AND CONCLUSIONS

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

Experimental

- Much laboratory data on shear modulus and damping ratio are presented. The dynamic shear modulus measured in different laboratory testing devices and under different types of loadings appeared to be compatible. The modulus reduction curves with the strain level seemed to be represented well by a modified Ramberg-Osgood two parameter equation of the form:

$$\frac{G}{G_{max}} = \frac{1}{1 + \alpha\gamma^{\beta}} \qquad (3)$$

  where G_{max} is the maximum shear modulus at $\gamma<10^{-4}$ %, and a and b are material constants. This corresponds with an equivalent viscoelastic stress-strain model and may be appropriate for shear strains $<10^{-2}$ %. For shear strains larger than this, shear volume coupling effects arise and must be considered.

- Cyclic testing under a loading sequence that more closely represents the field conditions is necessary to allow development of more rigorous stress-strain models..
- Data on lateral stress during undrained Ko shearing and subsequent consolidation suggest lateral total stress changes with excess pore pressures during earthquake loading.
- Static shear stress bias in one direction on a soil-structure interface could significantly affect the cyclic shear behaviour of the interface in the other direction.

Analytical Methods and Applications

- There is a need for a relatively simple elastic plastic model that captures both the shear and volumetric response of the soil skeleton under cyclic loading. A few plasticity models were presented that incorporate shear-volume coupling effects; however, these models appear to be very complex. The papers in this area are interesting and need further verification with laboratory element test data as well as centrifuge test data.
- The residual strength of soil is very important when investigating the possibility of a flow slide. Various methods of predicting cyclic shear strengths were evaluated by Chang and Hwang. Laboratory testing on high quality samples gave significantly higher values than in-situ test-based empirical methods. However, since the in-situ values are based on field experience, it is not clear that those higher values can be relied upon. It may be that the cyclic loading from the earthquake is more severe due to two and three dimensional effects. In addition, the undrained strength from triaxial compression loading may not be representative of field loading conditions.

TOPICS FOR DISCUSSION

Experimental

- There is a lack of laboratory research effort on evaluating dynamic properties of gravelly soils although there is a need for a better understanding of these materials. This is probably because much larger testing facility are needed to deal with larger particle sizes involved. However, we considered this area is worthy of attention.
- Are bender element applicable in gravelly soils? This is particularly relevant as dynamic shear modulus measurements of gravelly soils in resonant column tests are very difficult. Also, can bender element be calibrated to provide measurement of shear modulus and damping ratio at different strain level?
- Should we continue to concentrate on in-situ measurement of shear modulus and damping ratio, preferable at different strain level? There seems to be continued interest in attempting to obtain in-situ measurement of soil damping. However, separating material damping and radiation damping is a problem.
- In soil liquefaction, past efforts have been concentrated on evaluating the triggering resistance and residual strengths of clean and silty sands. We seem to have a reasonable understanding of the principles and various factors affecting the triggering of liquefaction. What we need now, is a better understanding of how the soil stress-strain behaviour changes from a stable state (pre-liquefaction) to a liquid state and its post-liquefaction stress-strain behaviour. Such information is crucial for a complete analysis to capture post earthquake displacements.

Analytical Method and Applications

- Retrofitting of existing structures where liquefaction is a concern requires an assessment of whether liquefaction is triggered or not, but also the prediction of the post triggering response. Such analyses would ideally be coupled effective stress analyses, but simpler uncoupled analyses to first predict the zones in which liquefaction is triggered, followed by simple deformation analyses based on post-liquefaction stress-strain curves would likely suffice in practice. This is so because the strains to trigger liquefaction are generally less than 1%, while the post-liquefaction strains could well be in the order of 30%.
- Verification of both the conventional uncoupled total stress analysis approach as well as coupled effective stress analyses against both centrifuge and field data is required to validate procedures.