04 Apr 1995, 10:30 am - 12:00 pm

Discussions and Replies — Session I

Multiple Authors

DISCUSSIONS AND REPLIES SESSION I

Discussion on paper titled: "Simple Shear Versus Direct Shear Tests on Interfaces During Cyclic Loading", by K. Fakharian and E. Evgin, Paper No. 1.05

By Muniram Budhu, Department of Civil Engineering & Engineering Mechanics, University of Arizona, Tucson, Arizona.

The authors described load-deformation characteristics for a sand-steel plate interface from monotonic and cyclic direct shear and simple shear tests. They showed that direct shear tests and simple shear tests gave approximately the same load-shear displacement and vertical displacement-shear displacement responses. These results are contrary to the findings of other researchers.

Why does the simple shear test give approximately the same results as the direct shear test? The authors did not address this question. Is the plane of failure and the stress state the same in both apparatus? The direct shear test forces the specimen to fail along a horizontal plane but the simple shear test does not. Stress and strain distribution in these two types of apparatus are also different. It is therefore very surprising that the two apparatus can give the same results unless the authors have succeeded in forcing the specimens in both devices to fail along the same plane, most probably the horizontal plane.

The authors prepared the specimens of sand in the two apparatus by pluviation followed by suction - presumably this means that excess sand was removed by vacuuming. This technique cannot guarantee a level sample surface and, from the discusser's experience (Budhu, 1979), suction leaves a loose layer of sand at the top of the specimen. This loose layer forms the interface between the specimen and the top boundary. Pluviation onto a surface formed by sand paper also leaves a loose layer at the bottom of the specimen (Cole, 1967; Stroud, 1971). The loose layers tend to entice failure along the interface as demonstrated by X-radiography (Budhu, 1979).

Reference

Budhu, M. (1979) Simple shear deformation of sands, Ph.D. Thesis, Cambridge University, UK.

Cole, E.R. (1967) The behavior of soils in the simple shear apparatus, Ph.D. Thesis, Cambridge University, UK.

Stroud, M (1971) The behavior of sand at low stress levels in the simple shear apparatus, Ph.D. Thesis, Cambridge University, UK.

Discussion on paper titled: "Study on Cyclic Shear Strength of Soils from Different Methods", by Chi-Tso Chang & Jin-Hung Hwang, Paper No. 1.09

By Muniram Budhu, Department of Civil Engineering & Engineering Mechanics, University of Arizona, Tucson, Arizona.

The authors presented a comparison between the shear strength (τ) to the effective vertical stress (σ'_v) ratios estimated from procedures to determine liquefaction potential of sandy soils with laboratory test results. Three methods were examined - SPT - N [Seed (1983, 1984, 1987), Japanese Bridge Design Method (JBD), Tokimatsu and Yoshimi method (1983)], CPT method [Sibata et. al. (1988)], and the seismic - v_s method [Tokimatsu et. al. (1990)]. They conducted cyclic triaxial tests on tube and block samples of silty sand extracted from a site at Peikang, Taiwan.

The authors showed that none of the methods agree with each other or with the tests results. This is not surprising. Seed's correlation curve of τ/σ'_v versus SPT-N is best used with simple shear test results rather than triaxial test results. Comparison of the initial stress state between simple shear and triaxial test (e.g. Castro, 1975) revealed that simple shear test results could be as much as 50% less than triaxial test results. Further, Seed and Peacock (1971) showed that expected free field values of τ/σ'_v are about 20% higher than laboratory simple shear values; indeed, one can expect differences between 15-50%.

The authors' results showed the dilemma faced by engineers who wish to determine the liquefaction potential of a site. Which method should be used? Seed's (SPT-N) method as shown by the authors gives lower τ/σ'_v values than the other methods for depths less than 10m while the JBD method gave the lowest τ/σ'_v values for depths greater than 10m. Each of the methods was developed for certain soil types and one cannot expect them to be reliable for all soil types.

Finally, no new significant finding was revealed in this paper. It is well known that there is no reliable correlation among SPT, CPT, wave propagation tests and laboratory tests.

Reference

Castro, G. (1975) Liquefaction and cyclic mobility of saturated, sands, ASCE, Vol. 101, No. GT. 6, pp. 551-569.

Seed, H. B. and Peacock, W.H. (1971) The procedure for measuring soil liquefaction characteristics, JSMFD, ASCE, Vol. 97, No. SM 8, pp. 1099-1119

Discussion on paper titled: "Study on Cyclic Strength of Soils from Different Methods", by C. T. Chang and J.H. Hwang, Paper No. 1.09.

By: Tej B.S. Pradhan, Dept. of Civil Engineering, Yokohama National University, Japan.

The authors have compared the cyclic shear strengths of soils predicted by different methods as SPT-N, CPT-q_c, Vs and undisturbed samples for the establishment of liquefaction criteria. The writer would like to add some comments on the following points that should be clarified more.

K_0-value: In predicting the field cyclic strength $(SR)_f$ from triaxial strength, K_0 from DMT test, has been used. What K_0 value was used? A larger $(SR)_f$ might be predicted since some data showed that K_0 from DMT have a tendency to give larger value as compared to laboratory tests. It also seems necessary to check how much difference does the value $0.9*(1+2K_0)/3$ makes with the correction factor C_r (=0.57) as suggested by De Alba et al. (1976).

Fines content: Generally it is said that fines content(FC) in sand increases the liquefaction potential. However, if the plasticity of the fines is very low, it has been reported that the liquefaction potential is lowered if FC< 20% for sands. Also, some data showed that increase in liquefaction potential cannot be expected for FC less than 15% for different kinds of soils. The writer believes that cares should be taken when using empirical relations for the effect of fines content on liquefaction potential.

Some discrepancy on using FC and D_{50}: Different liquefaction potential predicting methods use either FC or D_{50}. There are some defects pointed out by many researcher on these facts. For example, JBD, which uses D_{50}, tends to result in low liquefaction potential for soils with higher FC. Seed's method and CPT tends to result in low liquefaction potential for soils with high FC and low N value.
Recent great Hanshin earthquake (January 17th, 1995, M7.2) showed that soil (well graded decomposed granite) with D_{50} of about 3mm and FC of about 10%, liquefied intensively in a man made island in Kobe. It seems that liquefaction assessment on the insitu soil should be carried out at any seismic site irrespective of the liquefaction criteria.

Strengths of undisturbed samples: The authors have stated that the block samples (by insitu freezing) of loose sand layer gave high cyclic strength as compared to tube samples. The writer would like to know the insitu relative density (D_r) if measured. The reason is that, sometimes tube sampled specimen exhibit higher strength as compared to block sample due to the contraction caused by negative dilatancy when pushing the tube.

How much was the difference between the cyclic strength of reconstituted specimen (reconstituted at the insitu density) and the undisturbed specimen?

Discussion on paper titled: "Soil Damping and Its Use in Dynamic Analyses", by A.K. Ashmawy, R. Salgado, S. Guha, and V.P. Drnevich, Paper Nº 1.13.

By: F. Rodríguez-Roa, Professor of Geotechnical Engineering, Catholic University of Chile, Chile.

The authors have presented a very interesting and complete retrospective view on the use of soil damping in dynamic analyses. However, their conclusion from equation (10) that the damping ratio, D, used in most geotechnical engineering applications is an "equivalent damping ratio for a KV SDOF system at resonance", requires an additional consideration. It is known that the most accepted definition of damping ratio in soil dynamics is given by: $D = \Delta W/4\pi \cdot W$, in which ΔW is the area of the hysteresis loop, and W is defined as the area of the triangle OCC' (see enclosed figure) (Seed and Idriss, 1970; Ishihara, 1986). But the authors define W as the area of triangle ABC (figure 1), so for isotropic soils, subjected to symmetrical loading cycles, the difference in damping ratio would be 4 times, if we considered one definition or the other.

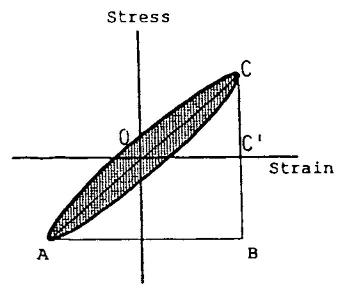


Figure. Hysteresis stress-strain loop

References

Ishihara, K. (1986), "Evaluation of Soil Properties for Use in Earthquake Response Analysis", Geomechanical Modelling in Engineering Practice, edited by R. Dunger and J.A. Studer, A.A. Balkema, 241-275.

Seed, H.B., and Idriss, I.M. (1970), "Soil Moduli and Damping Factors for Dynamic Response Analyses", Report Nº EERC 70-10, U. of California, Berkeley.

Discussion on paper titled "Soil Damping and its Use in Dynamic Analysis", by A. K. Ashmawy, R. Salgado, S. Guha & V.P. Drnevich, Paper No. 1.13

by: Diego Lo Presti, Department of Structural Engineering, Politecnico di Torino, Italy.

The paper points out the complex nature of damping in soils which involves both viscous phoenomena of the pore fluid and viscous and/or plastic phoenomena concerning the soil skeleton.
The authors therefore suggest of reconsidering the influence of rate of loading (and/or frequency) on soil damping. This suggestion which could have significant influence on seismic analysis of soil deposits for the following reasons:
- The authors have shown that damping significantly influences the response of a SDOF KV system for frequencies ranging from 0.5 to 2.0 times the natural frequency of the system.
- It is widely acknowledged that natural frequencies of soil deposits typically range from 1 to 10 Hz.
- Loading frequencies typically used in the laboratory fall in the following intervals:

0.1 - 1 Hz	Cyclic tests
40 - 200 Hz	Resonant Column tests
> 200 Hz	Seismic tests in the lab.

On the writer's opinion only the cyclic tests operate at frequencies or loading rates that are close to those encountered in soil deposits subject to seismic motion.

The authors show interesting experimental data concerning the influence of "disturbance" on damping ratio of a reconstituted kaolin specimen, tested in the RC apparatus. The authors show that at small strains damping ratio values do not change for effect of the disturbance, while at larger strains they significantly decreased. Moreover at small strain G_{max} showed a 50 % reduction upon disturbance.

The authors do not give information about the large strain shear modulus before and after disturbance.
However it should be also considered that a reduction of G_{max} of about 50 % involves a reduction of the resonant frequency at about 70 % of the values measured before the disturbance. The impact of a reduced loading frequency on D, as already pointed out, is still not clear, which limits the effectiveness of the authors' conclusion on this point.

Discussion on paper titled: "Soil Damping and its Use in Dynamic Analyses," by A.K. Ashmawy, R. Salgado, S. Guha & V.P. Drnevich, Paper No. 1.13.

By: C.T. Chang & J.H. Huang, Sr. Engineers, Sinotech Engineering Consultants, Ltd.

The authors provide an intensive review of various soil damping models and conclude that the frequency dependent visco-elastic model may better represent the behavior of soils. We intend to reserve this conclusion. From our profound seismic ground response analyses, the results show that the analyses using frequency dependent viscous damping lead to wide discrepancy of response of ground motion as compared with field measured results. This may attribute to filtering of high frequency content by viscous damping and therefore yield poorer results.

Generally, quasi-linear stress-strain relationship using frequency independent hysteretic damping can be used to obtain satisfactory results, e.g., SHAKE. In SHAKE analysis, some special techniques are still needed to get reasonable results. For instance, when convolution is carried out to get ground surface motion from the bottom of strata, the high frequency content has been overdamped. On the other hand, when deconvolution is carried out based on the ground surface control motion to get motion at the bottom of strata, the high frequency response is over-amplified as compared with the measured ground motion. Under such circumstances, cut-off frequency technique is employed to get rid of unreasonable results. Therefore, we considered that the hysteretic damping ratio will meet the damping behavior in the low frequency ground motion. As to what kind of damping will meet the behavior of high frequency ground motion can not be concluded presently, it requires more investigation.

Discussion on paper titled: "Modelling of Cyclic Behaviour of Sand in Large Range of Strain", By P.Y.Hicher and M.Kordjanil, (Paper No.1.17)

By: Yasuo TANAKA, Dept of Civil Eng., Kobe University, Nada, Kobe, JAPAN

The authors presented a numerical model for liquefacton of sand and also presented a verification of their model by comparing their prediction with available experimental data.

The authors write that an improvement of their prediction is made by adjusting the values of parameters r_{hys}, r_{mob}, a_c, and m. The improvement of their prediction is therefore not from the modification of their model, but rather from the changes of model parameters.

Although the predictions as depicted in Figs. 6 to 11 show a good agreement with the test data, it is not clear how much improvement is made from the previous prediction. Therefore, the results of the previous prediction needs to be presented. The discusser also believes that some explanation is needed on what are the physical meanings of the model parameters, r_{hys}, r_{mob}, a_c, and m, and what are the changes in these parameters.

Also as to the yield function of their model, an isotropic hardening model is used for analysing the sand behaviour of cyclic mobility. The discusser doubts about the applicability of isotropic model for such large deformation behaviour, and therefore thinks that the improvement by merely adjusting the model parameters will have a limitation.

Discussion on paper titled: "Nonlinear Cyclic Stress-Strain Relations of Soils" by K. Nakagawa, et all. Paper No. 1.22.

By: C.T. Chang & J.H. Huang, Sr. Engineers, Sinotech engineering Consultants, Ltd.

The authors propose the following model:

$$\frac{G}{G_{max}} : \frac{1}{1+\alpha|\tau|^{\beta}}$$

which is a two parameter model, α, β (G_{max} is not considered as a parameter). this model will have the similar modelling capacity as compared with Ramberg-Osgood model. It would be more practical if an expression could also be available for the damping value. We ever proposed a modified hyperbolic model as follows:

$$\frac{G}{G_{max}} : \frac{1}{1+\left(\frac{\tau}{\tau_y}\right)^n}$$

in which τ_y denotes the strain at yielding and can be obtained from $G / G_{max} \sim \log \tau$ curve where G starts degrading. Our model is quite similar to Nakagawa's model and can better define the physical interpretation of τ_y.

Discussion on paper titled "Nonlinear Cyclic Stress-Strain Relations of Soils", by K. Nakagawa & K. Soga, Paper No. 1.22

by: Diego Lo Presti, Department of Structural Engineering, Politecnico di Torino, Italy.

The paper presents a cyclic torsional shear apparatus developed in order to investigate cyclic (or equivalent G_{eq}) soil stiffness in the strain interval from 10^{-4} % to 1 %. A simple mathematical model to account for soil non linearity is also proposed.

The use of solid cylindrical specimens with variable shear strains from the centre to the edge of the circular section is, on the writer's opinion, a limitation, which can be easily overcome by using hollow cylindrical specimens. Moreover, as far as the test apparatus is concerned, it is not clear which kind of control is usually operated during a test (constant frequency, constant strain or stress rate).

The proposed model seems very simple and flexible. The authors have determined the model parameters (α, β) from experimental data published in literature as well as from their own experimental results. They showed that α and β are linked to each other. It is therefore possible to conclude that this model is completely determined by the knowledge of a single parameter which in turn seems to be dependent on the plasticity index (PI), as shown by the authors. In particular at a given strain level the G_{eq} / G_{max} ratio increases for increasing PI, as can be easily verified by using the values of α and β of figure 8.
This finding is in good agreement with what shown by Vucetic and Dobry (1991).
However the above conclusion is subject to some criticisms based on the following considerations:
- The major part of the experimental results, used by the authors and by Vucetic and Dobry (1991), were obtained in Resonant Column tests, which involve very high strain rates. Moreover in RC tests the average strain rate increases with increasing strain level. This consideration hold for constant frequency tests, too.
- The strain rate dependency of soil stiffness increases with the strain level, being almost negligible at small strains and becoming more and more relevant at large strains. The G / G_{max} vs. γ curves are therefore strain rate dependent.
- The strain rate effect on stiffness is of course more pronounced in soils with higher PI.

The dependence of α and β parameters on PI could therefore be a consequence of what above exposed.

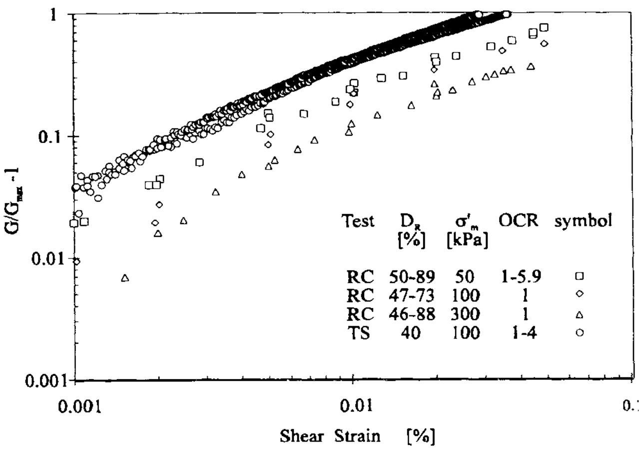


Figure 10 $(G/G_{max}-1)$ vs. γ

The writer have tested the proposed model with his own experimental data obtained on reconstituted Ticino sand specimens (figure 10). It is an uniform, medium to fine silica sand not containing fines. Experimental results were obtained from static monotonic torsional shear tests (TST) and from Resonant Column tests (RCT). The linear relationship of $(G/G_{max}-1)$ vs. γ in log scale, predicted by the model, is not exactly verified by the experimental results shown in figure 10. This confirm, on the writer's opinion, the need for models with variable parameters such as that proposed by Tatsuoka and Shibuya (1992).
It is also possible to observe that for this reconstituted sand the shear modulus reduction curve depends only on the confining stress level and type of loading. Infact different $(G/G_{max}-1)$ vs. γ curves are obtained in the case of static monotonic loading tests (TS) and cyclic dynamic tests (RC). This last observation makes questionable the use of the Second Masing Law.
The possibility of using the proposed model in conjunction with Masing criteria in order to predict material damping is only suggested by the authors but not verified with the available experimental data.

REFERENCES

Vucetic M. and Dobry R. (1991) "Effect of Plasticity on Cyclic Response" Journal of Geotechnical Engineering, Vol. 117, 1:89-107

Tatsuoka, F. and Shibuya, S.,1992,"Deformation Characteristics of Soil and Rocks from Field and Laboratory Tests," Keynote Lecture, IX Asian Conference on SMFE, Bangkok, 1991, vol. 2, pp. 101-190.

Discussion on paper titled: "Stress Dependence of Sand Stiffness", By D.C.F.Presti, M.Jamiolkowski et al., (Paper No.1.32)

By: Yasuo TANAKA, Dept of Civil Eng., Kobe University, Nada, Kobe, JAPAN

The authors are to be congratulated for presenting a very precise experimental data on soil stiffness at very small strain and these are very valuable indeed in understanding the fundamentals of sand deformation properties.

One of the purposes of their paper seems to be to examine the anisotropic deformation properties as developed with the increase of strain and the authors argument is based on the relationship between E from triaxial testing and G from torsional testing as their Figures 12 and 13 indicate.

The discusser accepts that a comparison between the small strain E and G does indicate the difference of stiffness on vertical and horizontal directions of the specimen initially prepared, but he questions on the validity of extending the same comparison for the data at larger strains. Because the triaxial and torsional tests induce different modes of deformation on the initial specimen, the structures developed at some level of strain would be different.

Therefore the examination on anisotropic stiffness as developed with strain may be better made by performing unloading-reloading test at different levels of strain under the same mode of deformation and by comparing the strain responses of the specimen in horizontal and vertical directions.

Discussion on paper titled: "A New Method for Determining the Anisotropic Parameters of Materials Under True Triaxial Cyclic Loading", By Q.J.Yang and B.Shackel, (Paper No.1.33)

By: Yasuo TANAKA, Dept of Civil Eng., Kobe University, Nada, Kobe, JAPAN

The discusser believes that the value of developing a mathematical analysis tool for soil behaviour is very identical to devising a test apparatus to obtain mechanical properties of soil. The true value of the tool would be most appreciated by showing convincing examples of test data which give a deep insight of soil behaviour.

The authors presented a mathematical tool to obtain anisotropic deformation parameters and showed examples of calculation in Appendix A based on an idealized data of true triaxial testing which is a very specialized devise not commonly available for routine testing.

If a true appreciation is to be made on the value of their analysis method, some comparison would be needed on the analyzed anisotropic parameters which are obtained from the true triaxial tests and the conventional triaxial test data using actual soil materials.

Discussion on paper titled: "Preshearing Effect on Effective Stress Paths", By Q.J.Yang, (Paper No.1.34)

By: Yasuo TANAKA, Dept of Civil Eng., Kobe University, Nada, Kobe, JAPAN

The author presented experimental data regarding the pore water pressure response of saturated clay under repeated loading. The main point of author's paper seems to be the changes in the pore water pressure response of the clay as the clay is sheared without previous cyclic loading history to the case with precious cyclic loading. The author also argues that the analysis of such pore water pressure response of clay will lead to a better modelling of foundation clay which is subjected to repeated loading.

However, in order to produce a soil model for such engineering analysis and assess the safety of structure on the foundation, the deformation properties of soil are essential. Without the deformation data, the value of experimental work will diminished. Therefore, presentation is needed on the deformation properties of the clay tested with respect to different types of pore water pressure response due to the cyclic loadings.

Discussion on paper titled "Evaluation of Bender Elements for Use with Coarse-Grained Soils", by S. Nazarian & S.S. Baig, Paper No. 1.35

by: Diego Lo Presti, Department of Structural Engineering, Politecnico di Torino, Italy.

The paper compares the small strain shear modulus (G_{max}) obtained from Bender Element (BE) and Resonant Column (RC) tests, performed on different kinds of medium to coarse uniform sands. Limitations and repeatability of BE tests are mainly concerned by the authors.
The authors have observed a better repeatability in the case of RC tests performed on C_A sand and glass beads. This is also the writer's experience, as well as that of other researchers (Lo Presti and O'Neill 1991, Hameury 1994, Fioravante et al. 1994): G_{max} values determined from seismic tests are more scattered in comparison to those obtained from RC tests, especially in coarse soils. For example, Hameury (1994) shows that a deviation of about ± 5% is typical for BE tests, in the case of a subangular, well graded, coarse to medium crushable sand (Quiou sand), while, for the same soil, the typical deviation observed in RC tests, is about ± 2 %.
The greatest source of uncertainty in seismic measurements is identification of time arrival. Of course, errors in the travel time determination become more and more relevant as the travel path length decreases.
Authors have also shown that $G_{max}(BE)>G_{max}(RC)$ in the case of M_R sand of about 20 %.
Published data on coarse and fine grained soils (Brignoli and Gotti 1992, Hameury 1994, Jamiolkowski et al. 1994) clearly indicate that $G_{max}(BE)>G_{max}(RC)$. Some of these data are summarised in Table 1.
Data in Table 1 seem indicate an increase of the ratio $\frac{G_{max}(BE)}{G_{max}(RC)}$ for increasing PI or N_G. Larger value of G_{max} observed in BE tests could probably be due to both the following factors:
- The increase of stiffness for decreasing strain level which probably occurs, in the case of soft uncemented clays, even at very small strains, within the so-called "elastic zone". It is supposed that the strain levels occurring in BE tests could be at least one order of magnitude smaller than those observed in RC tests.
- The increase of stiffness with increasing strain rate, which probably occurs, in the case of very high strain rates, even at very small strains. Some sandy soils seem particularly sensitive to the strain rate effects. It is supposed that, the higher frequencies involved in BE tests could induce greater strain rate in comparison to the RC tests.

Table 1 Ratio of the shear modulus determined with different methods in various soils

Soil	$\frac{G_{max}(BE)}{G_{max}(RC)}$	N_G %	PI %
Hostun sand	1.03	not available	-
Quiou sand	1.09	5.3	-
Pisa clay	1.25	13-19	23-46
Avezzano silty clay	1.10	7-11	10-30
Pontida silty clay	1.16	not available	11
Kaolin	1.16	not available	25

where: PI is the plasticity index and N_G is the normalised increase of the small strain shear modulus per log cycle of time which usually occurs during drained creep.

REFERENCES

Brignoli E. and Gotti M. (1992) "Misure di velocità di onde di taglio in laboratorio con l'impiego di trasduttori piezoelettrici" Rivista Italiana di

Fioravante V., Capoferri R., Hameury O. Jamiolkowski M. (1994), "Deformational Characteristics of Uncemented Carbonate Quiou Sand", IS Hokkaido, 1:

Hameury O. (1994), Comportement des Sables en Petites et Moyennes Deformations" Memoire de These de Doctorat, ENTPE, France

Jamiolkowski M., Lancellotta R. and Lo Presti D.C.F. (1994) "Remarks on the Stiffness at Small Strains of Six Italian Clays," Theme lecture session 1a, IS Hokkaido, Pre-print volume, pp. 95-114

Lo Presti D.C.F. and O' Neill D. (1991) "Laboratory Investigation of Small Strain Anisotropy in Sand," Proceeding of the First ISOCCT1, pp. 213-224, Postdam, NY.

Discussion on paper titled: "Dynamic Properties of a Granular Soil by F. Rodriguez-Roa, G. Palma, Paper No. 1.52.

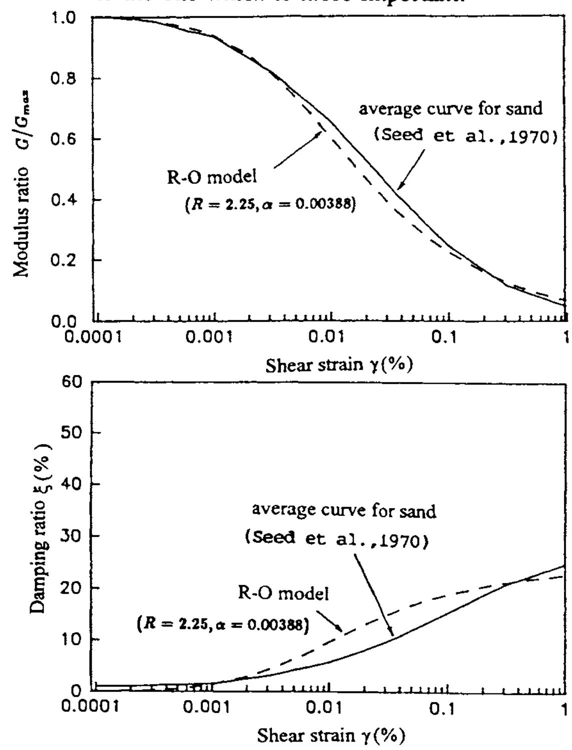
By: C.T. Chang & J.H. Huang, Sr. Engineers, Sinotech Engineering Consultants, Ltd.

The authors based on Masing rule formulate dynamic triaxial compression test and the result shows that the hyperbolic model is better than the Ramberg-Osgood model. This point needs to be further verified. It is noted that the Ramberg-Osgood model employs two parameters while the hyperbolic model uses a single parameter. If curve fitting technique is used, the former one appears better representative than the latter one.

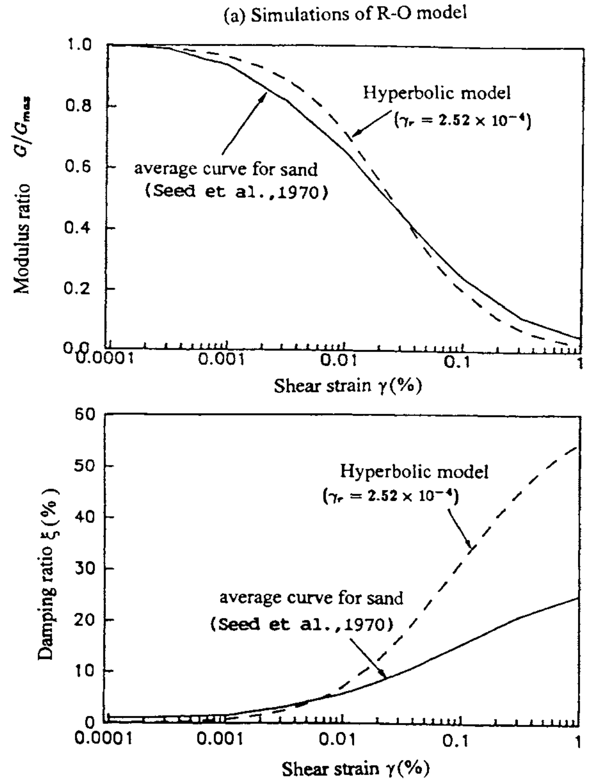
In G-τ and D-τ curves proposed by Seed et al. (1970), our study indicates that:

(1) Ramberg-Osgood model fits better than the hyperbolic model (as can be seen in Figure 1).

(2) No matter which skeleton model is used, Masing rule could not well represent G-τ and D-τ curves simultaneously. If the one fits well then the other fits poor. In practice, the better fit shall be applied to the one which is more important.



(a) Simulations of R-O model



(b) Simulations of Hyperbolic model

Figure 1 Comparison of simulations by R-O and Hyperbolic model

Paper No. 1.05
Reply by: K. Fakharian and E. Evgin, Department of Civil Engineering, University of Ottawa, Ottawa, Ontario, Canada

The authors appreciate Budhu's comments on the direct shear/simple shear testing of sand-steel interface. Budhu questions why the simple shear test gives approximately the same results as the direct shear test. The authors' paper demonstrated that no major difference exists between the results of two types of testing as far as peak and residual strengths of interfaces are concerned. However, when the initial stiffness (i.e. the slope of the load-displacement curve) is considered, the results obtained in two types of tests are different. The stiffness related to the total tangential displacement (i.e. the shear deformation of the sand mass plus the sliding displacement at the interface) has a lower value in the simple shear tests. On the other hand, the stiffness related to the sliding displacement alone is much larger in the simple shear tests. These results are in good agreement with those reported by Uesugi and Kishida (1986).

In simple shear testing of soils, failure was defined by Budhu (1988) as the maximum shear stress ratio mobilized on a plane which is not necessarily a horizontal plane. In the authors study, however, the failure occurs along the interface plane which is horizontal because the angle of friction between the steel surface and sand is lower than the friction angle of sand mass. As demonstrated in Fig. 2d, the shear deformation of sand mass in simple shear device, which is dominant before the peak, is negligibly small after peak point during which the sliding displacement or slip at the interface is increasing as shown in Fig. 2c. This shows that the failure has taken place at the interface and not within the sand mass. Since the stress state and the failure plane are the same in both testing methods, they provide the same strengths.

In the authors' experiments, very light vacuuming was used to level off the top surface of the sample. This surface is prevented from shearing due to the manner in which the soil containers and loading platen are designed (Fig. 1). Therefore, even if a very thin loose layer of sand is left at the top of the specimen, it does not interfere with the failure plane in neither direct shear nor simple shear soil containers.

Budhu also comments that a loose layer of sand might have been deposited at the bottom of the sample next to the interface during the air pluviation. The parametric studies carried out by the authors (Fakharian and Evgin 1993) indicated that there was a range of variations in the peak strength as a result of variations in the initial relative density of sand. If there was always a loose layer at the interface, the tests would show identical strength values irrespective of the density of the remaining soil mass. Therefore, the conclusions of the paper remain.

References

Budhu, M., 1988. The mechanism of failure under cyclic simple shear strain, Soil and Foundations, Vol. 28, No. 4, 119-129.

Fakharian, K. and Evgin, E., 1993. A three dimensional apparatus for cyclic testing of interfaces. Proceedings, 46th Annual Canadian Geotechnical Conference, Saskatoon, Canada, pp. 485-493.

Uesugi, M. and Kishida, H., 1986. Frictional resistance at yield between dry sand and mild steel. Soils and Foundations, 26(4), 139-149.

Paper No. 1.09
Reply by Chi-Tso Chang and Jin-Hung Hwang
Sinotech Engineering Consultants, Inc., Taipei, Taiwan

The writers thank the discussers for their comments on this paper. The discusser Pradhan, gives some valuable comments on the points of Ko-value, Fines content, Discrepancy on using FC and D_{50} and Strength of undisturbed samples. The Ko value was in the range of 0.50~0.72 from DMT test for the depths of 4m~20m and Ko=0.6 was used in the paper. The calculated correction factor 0.9x(1+2Ko)/3=0.66. There was about 15% difference of Cr as compared with that suggested by De Alba et al. (1976). The writers agree to the comment on FC and D_{50}, but do not agree to that the liquefaction potential is lowered if FC<20% for sands. As for recent great Kobe earthquake, (January 17, 1995, M7.2), the man made island although soil with D_{50} of about 3mm and FC of about 10% was liquefied intensively, it ought to be noted that the horizontal peak ground accelerations (PGAs) are unusually high (0.5~0.8g), which is beyond the data base to formulate the empirical liquefaction evaluation methods. The writers did not use the relative density Dr as a state parameter because e_{max} and e_{min} (maximum and minimum void ratio) are not reliably as determined in laboratory test for silty sands.

The triaxial cyclic strength SR_{20} (about 0.28~0.29) of reconstituted specimen (reconstituted at in-situ Dr=60%, FC<5%) is nearly the same as that of undisturbed specimen in this study.

The other discusser, Budhu, pointed out that triaxial cyclic strength should be corrected to field stress condition. The writers totally agree to this point and it was already taken into account by the correction factor 0.9x(1+2Ko)/3. The writers do not agree that each of the empirical methods was developed for certain soil types as pointed by the discusser because it is well known that these methods were deduced from wide range of data base. Finally, although the discusser said that everyone knows that there is no reliable correlation among these methods discussed in this paper, but it is still worthy to investigate further which method is more reliable and closer to actual observation data so as to enhance the state of arts in liquefaction evaluation.

Authors' Response to Discussions: "Soil Damping and Its Use in Dynamic Analyses," by Alaa Ashmawy, Rodrigo Salgado, Soumitra Guha, and Vincent Drnevich, Purdue University, W. Lafayette, IN, Paper No. 1.13.

Erratum: Equation (2) on page 36 should read:

$$\Psi = 4 \times \frac{A_{loop}}{A_{triangle}} \qquad (2)$$

where A_{loop} (=ΔW) is the area of the hysteresis loop, and $A_{triangle}$ (=4W) is the area of triangle ABC.

The authors would like to thank the discussers for their interest in the paper, and appreciate the comments and discussions received. They agree that much work needs to be done on understanding damping and its use in seismic response. For further insight to this issue, the readers are referred to the discussion by Drnevich and Ashmawy contained in this volume.

Discussion by Diego Lo Presti - The discussor suggests that frequency (strain rate) effects on damping were overlooked by the authors in forming Fig. 4 which showed the effect of disturbance on damping ratio. The authors do not believe that frequency effects need to be considered for these tests. To support this, Fig. 1 below gives the shear modulus as a function of shear strain amplitude. At high strain amplitudes, the shear moduli from the undisturbed and disturbed specimens converged and hence, the frequencies (and strain rates) were nearly the same. At low strains, the shear modulus for the undisturbed specimen was 50% higher than that for the disturbed. Accordingly, the frequency for the undisturbed case was approximately 120% of the frequency for the disturbed case. To cause significant variations in damping ratio, for undrained conditions at small strains, strain rates would have to be significantly different (by orders of magnitude) according to data presented in this conference by Tatsuoka et al. (SOA1, Figs. 46 and 49).

Discussion by C.T. Chang and J.H. Huang - The discussors are concerned that use of frequency independent damping in SHAKE analyses gives unreasonable results for high frequencies. The authors recognize this problem but believe that it is mainly due to the manner in which damping is applied within SHAKE. In SHAKE, the value of damping ratio for a given layer is selected on the basis of the maximum shear strain, and that selected damping ratio is applied to all frequency components, regardless of amplitude. Since higher frequencies are usually associated with low amplitudes, inordinately high damping is applied to these components. For further insight into the effect of daming type on soil amplification, the readers are referred to Paper No. 10.25 by Roesset et al, presented in this conference.

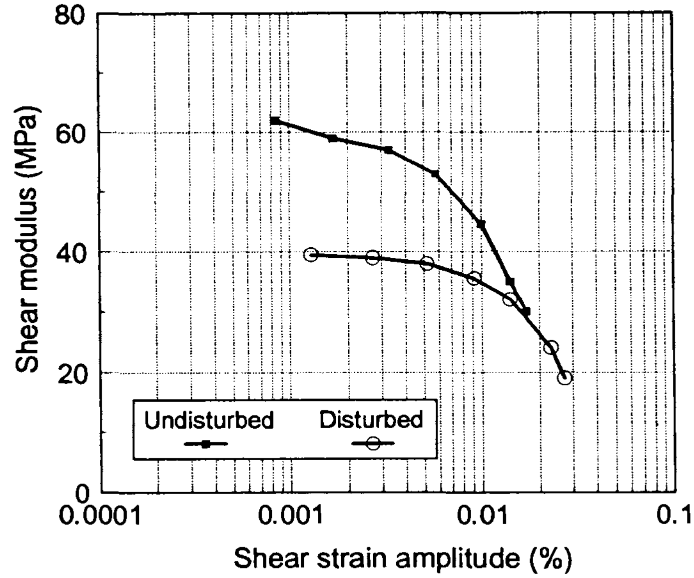


Figure 1. Variation of shear modulus with shear strain amplitude for kaolinite.

Discussion on paper titled: "Modelling of Cyclic Behavior of Sand in Large Range of Strain Amplitudes" by P.Y. Hicher and M. Kordjani. Paper No 1.17

by M. Budhu

Author's reply

Hujeux's Model is indeed a three-dimensional model: as presented in the paper, it consists of three plane-strain mechanisms in three orthogonal planes and of one isotropic mechanism. The purpose of this paper was not to demonstrate that the model was capable of reproducing liquefaction, which has already been done (Hujeux 1985), but to propose a methodology to determine the parameters, mainly those appearing in the cyclic hardening functions, in order to be able to predict the cyclic behaviour of a given soil. This was done in the paper by means of an inverse method, using the G-γ (or E-ϵ) decay curves, which are often available to describe dynamic soil properties.

The validity of the method was demonstrated by the agreement between computed results and test data in the case of a cyclic undrained test on a saturated sand leading to the liquefaction of the specimen.

Discussion on paper titled: "Modelling of Cyclic Behavior of Sand in Large Range of Strain Amplitudes" by P.Y. Hicher and M. Kordjani. Paper No 1.17

by Y. Tanaka

Authors' reply

The model does not contain only an isotropic hardening, but also a kinematic one in each deviatoric mechanism, using Mroz's concept of nested surfaces. The expression of the plastic modulus varies according to the position of the loading point in different domains limited by the values of r_{el}, r_{hyst} and r_{mob}. The physical meaning of these parameters referes to the concept of domains introduced by Ishihara (Hujeux 1985). For $r < r_{el}$, the soil is elastic; for $r_{el} < r < r_{hyst}$ the response starts to be non linear, but the cycle remains stable (small plastic strain, no pore pressure nor volume change during cyclic loading); for $r_{hyst} < r < r_{mob}$ the cyclic plastic strains become larger, pore pressure or volume change accumulates during cyclic loading; for $r > r_{mob}$ the soil undergoes large plastic strains which lead eventually to failure. We agree that more physical meaning should be put in these concepts. This can gradually be done by improving testing procedures, mainly in the domain of small strain (10^{-5} to 10^{-3}) and by theoretical studies in micro-macro mechanics.

The improvements of the predictions were mainly in a better adjustement of the number of cycles at liquefaction and a better agreement with the cyclic strain amplitude when liquefaction occured (see Hujeux 1985).

Authors' Response to Discussion: "Stress Dependence of Sand Stiffness", by Diego C.F. Lo Presti, Michele Jamiolkowski, Oronzo Pallara, Viviana Pisciotta and Salvatore Ture, Department of Structural Engineering, Politecnico di Torino, Italy, Paper No. 1.32

The authors would like to thank Prof. Tanaka for his interest in the paper and useful comments. The authors fully agree that the proposed experimental methodology is not the most appropriate way to investigate the stiffness anisotropy. On the other hand, if the experimental investigation is aimed to define the parameters of a cross-anisotropic medium, the use of a single specimen subjected to cyclic loading in Triaxial tests, as proposed by the discussor, is not able to give a complete information. Moreover, with a few exceptions, axial and radial strains are not measured in Triaxial tests with the same degree of accuracy.
As known a cross-anisotropic model is characterized by five independent deformation characteristics:

E_v = Young's modulus in the vertical direction
E_h = Young's modulus in the horizontal direction
ν_{vh} = Poisson's ratio for effect of the vertical stress on the horizontal strain
ν_{hh} =Poisson's ratio for effect of the horizontal stress on the complementary horizontal strain
G_{vh} = shear modulus referring to the vertical plane

With this respect the following experimental methodologies are, on the authors' opinion, more suitable in order to define the stiffness matrix of a cross-anisotropic medium:

1) The use of seismic tests propagating in dry or unsaturated soils both shear and compression waves (Stokoe et al. 1991, 1994, Lo Presti and O'Neill 1991, Bellotti et al. 1995). This method give only information on the small strain moduli.
2) The use of three different kind of tests (Lancellotta 1987) such as:

Compression loading triaxial test to obtain E_v and ν_{vh}
Torsional shear test to obtain G_{vh}
Plane strain extension tests to obtain E_h and ν_{hh}.
This method requires three duplicated specimens.
3) The use of torsional shear apparatuses with hollow cylindrical specimens having different inner and outer cell pressure which give the possibility to perform tests under a generalized stress state (Saada 1988, Wijewicreme and Vaid 1993). A lack of accuracy at small strains could be the main limit of this methodology.

The limits of each of these procedures have been briefly pointed out. The authors would like to conclude remembering that, when computing foundation settlements, the stiffness anisotropy becomes a factor to be accounted for only in the case that G_{vh} deviates significantly from the equivalent isotropic value $\frac{E_v}{2\cdot(1+\nu_{vh})}$ (Burland 1988). This is the only reason why figures 12 and 13 were presented.

REFERENCES

Bellotti R., Jamiolkowski M., Lo Presti D.C.F. and O Neill D. (1995) "Anisotropy of Small Strain Stiffness in Ticino Sand" Geotechnique (in press).

Burland J.B. (1988) "Recent Developments in Foundation Analysis and Design" Continuing Education Short Course, Imperial College, Department of Civil Engineering 12th to 14th April.

Lancellotta R. (1987) "Potenzialità delle Prove di Laboratorio" XIII CGT, Department of Structural Engineering, Politecnico di Torino, Italy.

Lo Presti, D.C.F. and O'Neill, D.A. (1991), "Laboratory Investigation of Small Strain Modulus Anisotropy in Sand", Proc. ISOCCT1, Postdam, NY

Saada A.S. (1988) "Hollow Cylinder Torsional Devices: Their Advantages and Limitations" ASTM STP 977,pp. 766-790.

Stokoe, K.H.II, Lee, J.N.K. and Lee, S.H.H. (1991), "Characterization of Soil in Calibration Chambers with Seismic Waves", Proc. ISOCCT1, Postdam, NY

Stokoe, K.H.II, Hwang S.K., Lee J.N.K. and Andrus R.D. (1994), "Effects of various parameters on the Stiffness and Damping of Soils at Small to Medium Strains" Keynote Lecture 2, IS Hokkaido.

Wijewickreme D. and Vaid Y. 1993, Behaviour of loose sand under simultaneous increase in stress ratio and principal stress rotation. Canadian Geotechnical Journal, 30:953-964

Paper No. 1.52

Reply by F. Rodríguez-Roa and G. Palma

The comparison carried out between the Hyperbolic Model and the Ramberg-Osgood Model was herein only extended to the analysis of shear stress-strain laws as it is mentioned in the paper.

Twenty-four different G-γ curves were examined. The results obtained showed that in 79% of the analyzed cases the Hyperbolic Model performed better than the R-O Model. When the Hyperbolic Model was applied the correlation coefficient ranged from 0.867 to 0.999, with an average value of 0.978. On the other hand, using the R-O Model the correlation coefficient varied between 0.822 and 0.999, with an average value equal to 0.952.

The degree of fitting of an empirical relationship not only depends on the number of parameters but also on the analytical expression itself. Nakagawa and Saga in their presentation to this Conference (Paper No. 1.22) propose a new law to express the degradation of Dynamic Shear Modulus with Shear Strain level. This relationship depends only on one independent parameter and its performance would still be better than both the Hyperbolic Model and the R-O Model, for the various soils tested.

Concerning damping ratios, the values herein included, were all directly obtained from the measured hysteresis stress-strain loops.