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06 Apr 1995, 10:30 am - 12:30 pm

Discussions and Replies — Session III

Multiple Authors

DISCUSSIONS AND REPLIES
SESSION III

Discussion on paper titled: "Application of Deep Compaction Techniques to Liquefaction Prevention", by G. Armijo, P. Sola and C. Oteo (Paper No. 3.01)

By: M.A. Mollah, Government Center for Testing & Research, Ministry of Public Works, Kuwait.

The authors concluded that vibratory techniques namely, dynamic compaction, vibroflotation and vibro-replacement (stone columns) can be effectively applied to most potentially liquefiable soils to prevent liquefaction.

The conclusion is based on data presented in Fig. 1 which shows grain size envelope of soils liquefied during some of the past earthquakes in Japan and Alaska, and also those treatable by the application of these deep compaction techniques.

The authors are to be commended for their excellent compilation of past works on application of vibratory techniques and presentation of empirical approaches which will serve as a useful reference for professionals involved in soil improvement. The writer, however, has the following comments:

1- For ground treatment, the authors recommend DGI of about 1.75 to 2.00 when $N_1 < 20$. Will a loose sand improved with this criterion be effective against occurrence of liquefaction?

2- The paper appears to be literature dependent. It would be illuminating if the authors share actual field experience highlighting the problems and difficulties encountered.

3- Proper reference for the data contained in Fig. 1 is desirable for possible utilization by others.

Discussion on Paper Titled: "Liquefaction Damage of Sandy and Volcanic Grounds in the 1993 Hokkaido Nansei-Oki Earthquake," by S. Miura, K. Yagi and S. Kawamura, Paper No. 3.06.

By: Takeji Kokusho, Central Research Institute of Electric Power Industry

The authors have shown that during the earthquake sandy soils with variety of particle gradings as indicated in Fig. 5 actually liquefied. They have also shown experimentally that undrained cyclic shear strength of volcanic or fine-grained sand exhibits much smaller strength than normal clean sand.

In order to offer some additional findings and comments to what they have provided in the paper, the discusser would like to address the following comments.

During the same earthquake, as introduced by the same authors, volcanic soils consisting of gravels, sands and silts have liquefied in the Akaigawa district near the Mori Port, inflicting differential settlements to more than forty houses there. A detailed study on this gravelly soil is available in the Paper 3.20 presented in this conference by the present discusser, which indicates:

a. Even a very well-graded gravelly soil of Uc=300 can be susceptible to liquefaction if it is very loosely deposited. Therefore, there seems to be no boundary to preclude liquefaction susceptibility as far as the particle grading curve is concerned.

b. One should be careful enough to avoid very erroneous conclusions on particle gradings of liquefied soils as depicted in the liquefaction case in Akaigawa district where only silty and sandy matrix came up to ground surface from very course soils down below the 70% of gravel content.

c. The undrained cyclic strength (ϵ_{DA}=2%, N=20%) of the gravelly soil, the relative density of which was as low as 20%, was evaluated as 0.18 based on large triaxial tests on intact samples of 30 cm in diameter. This strength should be properly interpreted in the light of the experimental results indicated by the present authors that volcanic sands exhibit much lower liquefaction strength than normal sands.

d. Through liquefiable, well graded soils have smaller difference between maximum and minimum void ratio, which indicates smaller potential for larger settlements and flow slides after the onset of liquefaction.

Discussion on paper titled "Effect of the Grain Size on the Energy per Unit Volume at the Onset of Liquefaction",
by A.S.Saada & L.Liang (Paper No. 3.07)

By: Eugene A. Voznesensky, Department of Engineering Geology, Moscow State University, Russia

The authors obtained, that (1) silty sand required lower energy per volume to reach liquefaction than clean sand, and (2) increase of silt content eliminated the influence of relative density on energy required for liquefaction. So, they conclude about the significant effect of silty particles, present in the inter granular space on the kinematics of cohesionless soil.

However, in this case we have not an effect of grain size decrease only. The two tested soils have different nature of inter particle bonding. ~~From Fig.1 it is seen that~~ the Lower San Fernando Dam silty sand contains about 3% of clay particles (less than 0.001 mm) that must cause weak thixotropic properties of soil. The fact that sand-clay mixtures (with about only 1.5-2% of clay) perform thixotropy to some extent has been demonstrated by Prof. P.G.Boswell (1949) and later studies of other researchers. Partial thixotropic regain was observed in our undrained triaxial tests of soils with very much alike grain-size composition (paper 3.53). These silty soils are not cohesionless ones and could be called "low-cohesive soils". Very low unit energy for liquefaction is really one of their characteristic features (and the discussed paper verifies it by original and interesting data). It is connected with different nature of their structural bonds: part of them are mechanical ones - due to Coulomb friction, and part - physico-chemical (coagulative) - due to van-der-Vaals forces. But friction in silty soils is less than in clean sands, and coagulative structural net - much weaker (and, most likely, discontinuos) than in plastic clays, both resulting in extremely instable and very sensitive to dynamic loading structure. It seems likely that unit energy for liquefaction should decrease from coarse to fine sands, but there may appear a noticeable drop of this energy at the sands/silty soils border.

Discussion on paper titled: "Liquefaction Potential Evaluation for the Messina Straits Crossing by Field and Laboratory Testing", by Pelli,F., Tokimatsu,K., Yoshimi, Y. and D'Appolonia, E. (Paper No. 3.13)

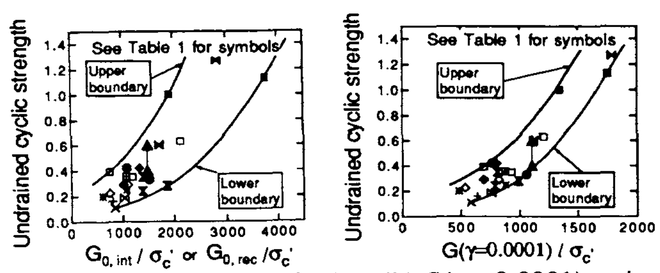
By: Yukihisa Tanaka, Central Research Institute of Electric Power Industry, Japan.

The authors conducted 'improved triaxial test' to evaluate in-situ undrained strength without the effect of sample disturbance. However I think that in-situ $G/G_0 \sim \gamma$ relation as well as in-situ shear wave velocity is necessary for accurate evaluation of in-situ undrained cyclic strength. The reason is described below.

Figure 1(a) shows the relationship between undrained cyclic strength and initial shear modulus of intact or reconstituted gravelly soil samples, where $G_{0,int}$ and $G_{0,rec}$ denote the initial shear modulus by the cyclic triaxial test for intact samples and reconstituted samples, respectively. In this discussion, 'intact sample' means a sample obtained by the in-situ freezing method, while the undrained cyclic strength is defined as the stress ratio required to reach the double amplitude of axial strain, 2% or 2.5%, in 20 loading cycles. Though there seems to be some correlation in Fig. 1(a), it is not sufficient for accurate evaluation of undrained cyclic strength.

Since undrained cyclic strength is a mechanical property at large shear strains, the shear modulus at relatively large strains may be more closely related to undrained cyclic strength than initial shear modulus(Tanaka et al. 1992). Fig .1(b) shows the relationship between undrained cyclic strength and $G(\gamma=0.0001)$, which is defined as the cyclic shear modulus of shear strain amplitude, $\gamma=0.0001$. $G(\gamma=0.0001)$ were calculated from the average initial shear modulus of the samples used for the undrained cyclic strength test and degradation curves of shear modulus obtained from the cyclic deformation tests. As expected, the scatter of the data in Fig. 1(b) was smaller than that in Fig. 1(a).

Figure 2 shows the relationship between reference strain of intact sample and that of reconstituted sample. Both reference strains are different each other. Thus $G/G_0 \sim \gamma$ relation of intact sample and that of reconstituted sample are different each other. Therefore, in-situ $G/G_0 \sim \gamma$ relation as well as in-situ shear wave velocity is necessary for accurate evaluation of in-situ undrained cyclic strength.



(a) $G_{0,int}/\sigma_c'$ or $G_{0,rec}/\sigma_c'$ (b) $G(\gamma=0.0001)_{,rec}/\sigma_c'$

Fig.1 Undrained cyclic strength vs. shear moduli

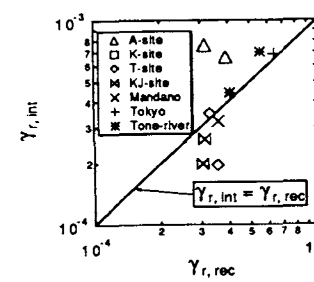


Fig.2. Comparison of reference strain of intact samples with that of reconstituted samples

Table 1 Symbols in Fig.1

Site		Sample quality	Layer	σ_c' (kPa) for cyclic strength test (for cyclic deformation test)	In-situ shear wave velocity, V_s (m/s)	Undrained cyclic strength	Symbol	Reference
A-site	Conducted in 1984	Intact	Upper	69 (54)	290	0.42	●	
			Lower	118 (104)	290	0.33		
	Conducted in 1987	Intact	Lower	98 (98)	290	0.4～0.6	▲－▲	
K-site		Intact		98 (98)	480	1.13	■	Tanaka, et al. (1991)
		Reconstituted		98 (98)		0.63	□	
T-site		Intact	Upper	176 (176)	405	0.42	◆	Tanaka, et al. (1992) Kokusho, et al. (1994)
			Lower	225 (225)	380	0.29		
		Reconst.	Upper	176 (176)		0.29	◇	
			Lower	225 (225)		0.225		
KJ-site		Intact	Upper	157 (157)	600	1.27	⧓	
			Lower	157 (157)	600	0.60		
		Reconstituted		157 (157)		0.187	⋈	
H-site		Intact	Upper	78 (78)	333	0.28	⧗	Tanaka, et al. (1994) Kokusho, et al. (1994)
			Medium	118 (118)	333	0.24		
			Lower	206 (206)	476	0.35		
Mandano Gravel		Intact		118 (118)	300	1.00	⊠	Goto, et al. (1987)
		Reconstituted		118 (118)		0.11	×	
Tokyo Gravel		Intact		294 (294)	380	0.355	⊞	Hatanaka, et al. (1988)
		Reconstituted		294 (294)		0.16	+	
Tone-river Gravel		Intact	Upper	78 (128)	330	0.35	⊡	Goto, et al. (1992)
			Lower	128 (186)	420	0.39		
		Reconst.	Upper	78 (128)		0.25	✳	
			Lower	128 (186)		0.20		

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Tanaka,Y., Kudo, K.,Yoshida,Y. and Kokusho,T. (1992). Undrained cyclic strength of gravelly soil and its evaluation by penetration resistance and shear modulus. Soils and Foundations, Vol.32, No.4, pp.12－142.

Discussion on paper titled: "Liquefaction during the 1991 April 22 Telire-Limon Earthquake and Correlations with the methods of Seed and Iwasaki", by P. Hafstrom, J. Skogsberg and A. Bodare (Paper No. 3.14)

By: M.A. Mollah, Government Center for Testing & Research, Ministry of Public Works, Kuwait.

The authors suggests using Iwasaki method for the assessment of liquefaction susceptibility of Cost Rica soils.

This suggestion is based on data presented in Table 1 where two well known empirical methods, Seed and Iwasaki, are employed to evaluate the liquefaction potential of sites within the area affected by liquefaction due to the 1991 April 22 Telire-Limon earthquake. The predictions are compared with incidents observed during the earthquake.

The writer congratulates the authors for such an informative paper and wishes to invite them to make comments on:

1- It lacks geotechnical data on soil condition, water table, regional geotectonic setting. Such information would have made the paper more comprehensive.

2- It would be interesting to know how the SPT N-value has been selected for calculation?

3- Is it not proper to calcuiate a-values? What is the basis of selection two a-values used in the calculation?

3- Attenuation formula obtained by personal communication from Tailor should have been produced for the benefit of others.

4- Zonation map mentioned in the paper is not found.

5- The severity values of 0 (no liquefaction), 1 (moderate liquefaction) and 2 (severe liquefaction) correspond to what ranges of calculated FS?

Discussion on

"A Procedure to Assess the Stability of Buried Structures"

by

M.Satoh, R.Isoyama, M.Hamada and A.Hatakeyama (Paper No. 3.17)

"Area of Compaction to Prevent Uplift by Liquefaction"

by

Y.Tanaka, H.Komine, J.Tohma, K.Ohtomo, H.Tochigi, H.Abo and S.Fukuda (Paper No. 3.34)

and

"Stress-strain Relationship of Sand after Liquefaction"

by

S.Yasuda, N.Yoshida, T.Masuda, H.Nagase, and H.Kiku (Paper No. 3.40)

by

I.Towhata
University of Tokyo, JAPAN

The writer has been studying the lateral movement of liquefied ground for the past decade. Related to this topic, the present discussion is hence concerned with the three papers mentioned above.

It is important that earthquake damages related to liquefaction are caused by deformation of ground. When the induced deformation exceeds the allowable limit, facilities cannot be used any more. The significant ground deformation exerts a large force to buried structures and could destroy them. Thus, it is of importance to somehow predict the probable ground deformation and assess the possibility of loss of facilities' operation.

An example is hereby taken of the distortion of a beam undergoing load. This distortion is normally calculated by using the well-known theory of structural mechanics which requires the followings to be known;

1) material property of beam, i.e. the bending stiffness EI,
2) type of support and boundary conditions; e.g., fixed or free.
3) load; magnitude and location.

No reasonable calculation is possible without knowing any of these.

In the case of liquefied ground that moves laterally, many studies have so far made. However, those studies have put emphases mainly on the material properties and have paid some attention to load, but the boundary condition has not been considered very much. Information needed for prediction of ground displacement is classified below;

1) material properties of liquefied sand; density, shear modulus, shear strength at large deformation, etc.
2) geometrical information; size of liquefied zone, thickness of liquefied layer, depth of ground water table, and boundary conditions (cracking or fixed).
3) load; unit weight of sand, surface gradient, intensity and duration of earthquake shaking.

The material property of liquefied sand is strongly affected by the density and the confining pressure. Tests by Verdugo (1992) in Fig.1 illustrates the stress-strain behavior of Toyoura sand undergoing CU triaxial compression. It is noteworthy that two specimens of the identical void ratio (0.908) exhibited completely different behaviors; under higher confining pressure, softening occurred after the peak strength, while continuous hardening was observed under the lower pressure. Since the free flow of liquefied ground under the gravity field is possible only when softening occurs, shaking table tests under low pressure level should pay a special attention to the density. Use of the same density as in the field is not recommendable. One of the solutions to this problem is a use of looser sand in shaking table tests; softening reproduced thus under low pressure in Fig.1.

Shear strength and, in particular, the undrained residual strength depends highly upon the sand density (Fig.1). Since the intensity of both dynamic and static stress is low in shaking table tests, the use of lower density reduces the strength and help reproduce in model tests the field phenomenon of flow failure. Model tests by Tanaka et al. employed approximately the same density as in-situ, the amount of ground deformation or uplift of a pipe may be underestimated.

Fig.2 illustrates the post-liquefaction deformation of a slope that was observed in a shaking table test. The magnitude of lateral displacement varied with the location. Since no cracking was allowed, the displacement was null at two ends of the container. The maximum displacement occurred near the middle of the slope.

The geometrical information plays an important role in prediction of displacement. Firstly, the size of a liquefied zone exerts a significant influence on the magnitude of displacement. A model test carried out in a small container can never develop such a large displacement of, e.g., 2m that exceeds the size of the employed container, although the same magnitude of displacement is often observed in case histories. Secondly, the location of a concerned site in a single liquefied zone affects the magnitude of displacement. In the past experiences, the displacement is greater towards the top of a slope, whilst it decreases near the bottom end. Thirdly, the slope inclination, together with the unit weight of soil, determines the intensity of static shear stress that is the load to cause the displacement. No significant displacement is likely in a uniformly level ground. Satoh et al. did not consider either the geometrical or load issues in their simplified formulae (Eqs. 1 and 2) for displacement prediction.

Yasuda et al. made an attempt to predict the lateral displacement of a slope by integrating what they call the reference strain beyond which sand starts to develop a significant shear resistance. The reference strain varies with the intensity and duration of earthquake loading. Thus, the material property is considered in their study. It is noteworthy, however, that the effects of slope inclination and size of liquefied zone are not taken into account. Either geometrical or load issues should not be neglected.

In summary, it is found that studies on prediction of large deformation in liquefied ground does not pay sufficient attention equally to three important issues; material properties, geometrical information, and load. When even one of them is neglected, no reasonable prediction of deformation is possible.

REFERENCES

Verdugo,T. (1992) "Characterization of Sandy Soil Behavior under Large Deformation," Ph.D.Thesis, University of Tokyo.

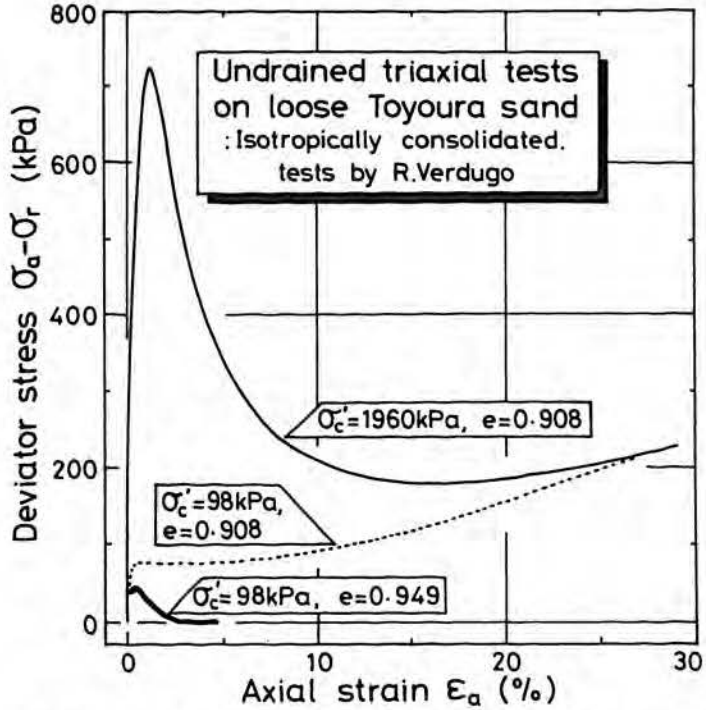

Fig.1 Effects of density and confining pressure on CU behavior of loose sand.

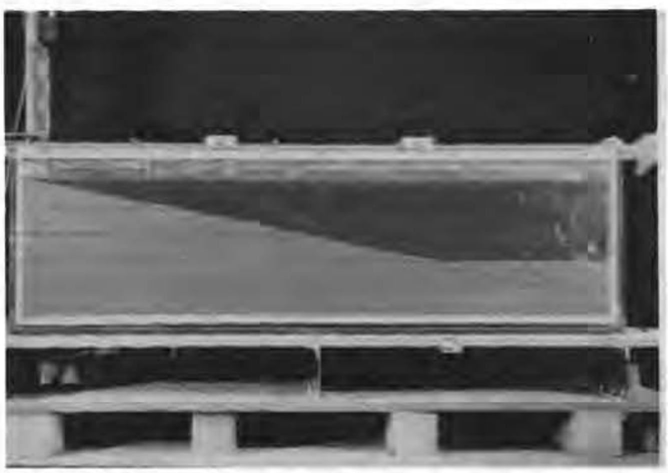

Fig.2a Shape of loose model slope prior to shaking.

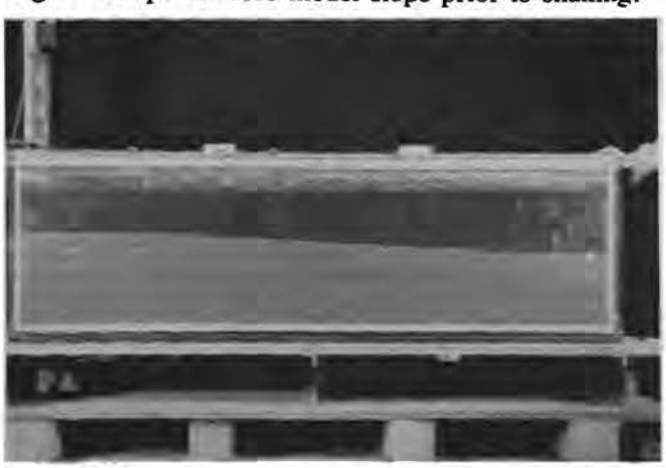

Fig.2b Post-liquefaction configuration of a loose model slope.

Discussion on paper titled: "Correlation between CPT Data and Dynamic Properties of In Situ Frozen Samples," by Y. Suzuke et al. (Paper No. 3.22).

By: James P. Lee, Senior Consultant, Brown & Root.

This paper provided several correlation charts for cyclic stress ratio based on CPT data. Figure 1 also includes a comparison of the SPT data with the CPT data. It would be very helpful for the comparison of the CPT and SPT results, if the correlation charts for the CPT include plots of the SPT data. Evaluation of liquefaction potential by engineers in the USA is mostly based on SPT data.

Discussion on paper titled "The Liquefaction of Sand Lenses Due to Cyclic Loading", by J.D. Holchin & L.E.Vallejo (Paper No. 3.30)

By: Eugene A. Voznesensky, Department of Engineering Geology, Moscow State University, Russia

1. It is obvious from the testing results that the reaction of sand lenses depends on the intensity of vibration and hence becomes noticeable after a certain duration of shaking.

But we cannot understand this phenomenon in detail unless pore pressure build-up is monitored, because thixotropic strength degradation of the overlaying plastic clay can contribute to crack propagation and to lens collapse to some extent.

One of the possible ways to measure pore pressuse in sand lens in this test configuration is to locate a thin electric cable not upward, but horizontally in the clay stratum and take it out through the carefully plugged hole in container wall. In this case the cable should be put into clay slurry prior to consolidation.

As soon as the authours needed a visual observation of lenses behavior, they had to locate the cavity just by the container wall. So, possible partial pore water drainage along this wall must be taken into consideration since the impermeability of clay plexiglass connection could not be perfect.

2. The authors state that sand in lenses was in relatively loose condition, but there is no relative density values. D_r could, however, be easily calculated, the volume of cavity being known.

3. It seems reasonable that two additional aspects should be taken into consideration for evaluation of sand lenses dynamic behavior. Firstly, it is their shape - higher stresses are concentrated at the tips of such a cavity with the decrease of its curvature radius according to Kolosov-Ingliss solution. And secondly, some sand lenses can contain confined (artesian) water.

Discussion on paper titled: "An Empirical Formula for Evaluation of Buildings Settlements due to Earthquake Liquefaction", by H. Liu (Paper No. 3.39)

By: M.A. Mollah, Government Center for Testing & Research, Ministry of Public Works, Kuwait.

The author proposes an empirical relationship involving earthquake intensity, contact pressure at foundation level and relative density of soil for quantitative evaluation of subsidence of liquefiable supporting layer.

The proposition is based on data presented in Table 1 which summarizes observed liquefaction subsidences from some past earthquakes of Japan and China.

The writer wishes to commend the author for presenting such an empirical formula for prediction of liquefaction subsidence and invite him to make comments on:

1- Table 1 should have included some details of in-situ soil condition of the sites and also the subsidence predicted under in-situ soil condition by the formula ($S_{predict}$). The comparison of observed and predicted subsidence could have been easier.

3- From the paper, it appears that the application of the formula is limited to sandy soils having a maximum fines content of 10%.

4 In Fig. 8, the shaded areas present the variation of S/De for Dr = 30-50%. The solid line which represents Dr = 50% should have formed the lower boundary?

Finally, will the author please care to correct Table 1 where last column is wrongly written as B/De instead of S/De?

Discussion on paper titled "Effective Stress Liquefaction Analysis at the Wildlife Site", by P.M.Byrne & J.McIntyre (Paper No. 3.49)

By: Eugene A. Voznesensky, Department of Engineering Geology, Moscow State University, Russia

The authors show that the proposed in their paper an incremental stress-strain model can be applied to predict dynamic response of granular soils in the field. But the measured pore pressure values are not in good agreement with the predicted ones: according to piezometer readings liquefaction took place some 30 sec later than it was predicted. The authors conclude that liquefaction could be triggered in some zones of the layer later than in the others.

This is, certainly, a possible explanation. But most probably this is due to another effect. Liquefiable layer at the discussed Wildlife site consists of silty sand, which is not a clean cohesionless soil. Presence of silt and even few (1-1.5%) clay particles may cause some thixotropy (their van-der-Vaals attraction forms a weak coagulative net), influencing dynamic response to some extent. So, rapid strength degradation began at about 17 sec, resulting in abrupt stiffness decrease (Fig.9b) at this very moment. But full destruction of coagulative net needs at least some seconds (since a certain quantity of energy should be absorbed by the soil) - so, complete strength degradation and subsequent liquefaction could really occur later - at about 50 sec.

Reply to Discussion on Paper #3.07
By J. Ludwig Figueroa

The influence of thixotropy on the liquefaction potential of soils is indeed an interesting subject. The authors do not know how much of it is present in a material with no apparent cohesion. Thixotropy is defined as an isothermal, reversible time-dependent process occurring under conditions of constant composition and volume whereby a material stiffens while at rest and softens or liquefies upon remolding. It is very difficult to find out if those symptoms are present in a mixture of sand and 20 percent silt. Time had certainly very little to do with our experiments where the specimens were prepared and tested within a short time interval. We believe that there is always a little gain in strength due to some kind of cementation that occurs when granular materials with fines are kept under pressure. We doubt, however, that this effect would have affected in a measurable way the energy needed for liquefaction in the case of our laboratory specimens.

Paper No. 3.13
Reply by F. Pelli, K. Tokimatsu, Y. Yoshimi and E. D'Appolonia

We thank Dr. Tanaka for his interest in our paper. Dr. Tanaka states in his discussion that the in situ G/Go vs. γ relationship as well as the in situ shear wave velocity is required to evaluate accurately the in situ undrained cyclic strength. He also provides some interesting experimental data, in the form of shear modulus vs. undrained cyclic strength diagrams, to support his point. The discussor clearly refers to the soil shear modulus as an indirect means to predict the soil resistance to liquefaction. On the other hand, the subject of our paper was the use of shear wave velocities to reconstitute laboratory samples for cyclic testing. In the following a few comments are presented with reference to both subjects.

1) Dr. Tanaka states that "since undrained cyclic strength is a mechanical property at large shear strains, the shear modulus at relatively large strains may be more closely related to undrained cyclic strength than to initial shear modulus". As a matter of facts the results presented by the discussor appear to confirm this statement. It is worth pointing out, however, that the liquefaction characteristics of soil as well as the soil shear wave velocity are generally recognised to be related to the initial state of the sand, as well as to its fabric and other fundamental parameters (e.g. grain size), which are independent from the strain level. The working principle of the (indirect) methods based on shear wave velocity measurements relies on the ability of this parameter to catch the (initial) key factors controlling the liquefaction process. This ability is currently a major subject of discussion. On the other hand, there is little doubt that the sand liquefaction process involves large deformations, and that testing at sufficiently large strain levels (e.g. undrained triaxial) is required for proper characterization.

2) Measuring the shear wave velocity is by far easier than obtaining a complete stress strain curve in the field. Since the seismic cone has been introduced on the market, shear velocity measurements have become nearly standard for offshore investigations in seismic areas. On the other hand, more complex field tests as required for soil characterisation at relatively large deformations are much more difficult to perform, if possible at all, in critical conditions such as in the Messina Straits (see Point 4). This aspect is quite important, as the main justification for indirect evaluations of the soil liquefaction potential resides in simplicity.

3) Indirect methods based on SPT, CPT or shear wave velocity, although most useful in many projects, are often not sufficient for the design of large offshore structures in highly seismic areas. For these projects, the structure deformation associated with earthquake-induced cyclic mobility must be determined with some degree of reliability. On the other hand, techniques such as soil freezing, which can provide good quality laboratory samples (e.g. Yoshimi et al., 1994), will not be applicable to offshore investigations at least until an efficient down-hole freezing system is developed.

4) The approach which was undertaken for the Messina Straits project (described in our paper), consisted of reconstituting laboratory samples to match the in situ shear wave velocity in order to re-create part of the original soil fabric. To this purpose, the methodology developed by Tokimatsu et al. (1986) was applied to the Messina Straits dense sand and gravel. The paper intends to show how this methodology proved useful in the offshore environment, where the recovery of "undisturbed" samples may be simply impossible in many instances (large water depths, strong water currents, difficult soil conditions with presence of gravel). Similarly difficult is the performance of relatively complex field tests which could in principle provide better understanding of the soil state.

5) The principle of reconstituting the samples based on the available field measurements, and of testing these "improved" samples in the laboratory, appears a viable and effective solution to explore the soil behaviour under cyclic loading. In those cases where recovery of good quality samples is possible, a comparison between field and laboratory measurements should be applied to verify the sample quality. Within this framework, the shear wave velocity is a very useful parameter for the following reasons: a) it is measured almost routinely offshore by seismic penetration probes, b) it can be easily measured in the laboratory as a non-destructive test for comparison purposes, c) it is affected by the soil fabric and by the initial state of the sand which also affect the soil resistance to liquefaction. It is interesting to note that in the petroleum industry the principle of comparing down-hole logs with laboratory sample characteristics has been proposed for several years for reservoir sands.

6) In addition to the shear wave velocity other quantities should be measured in the field. The void ratio, currently estimated based on CPT cone resistance or similar means, should be evaluated independently, based for instance on nuclear or resistivity methods, and more research is required in this field, particularly for offshore applications.

REFERENCES

Tokimatsu K., T. Yanazaki and Y. Yoshimi, 1986, "Soil liquefaction evaluation by elastic shear moduli", Soils and Found. 26, Vol.1, 25-35.

Yoshimi Y., K. Tokimatsu and J. Ohara, 1994, "In situ liquefaction resistance of clean sands over a wide density range", Géotechnique 44, No.3, 479-494.

Reply to Discussion on Paper #3.30
By J.D. Holchin

The Author appreciates the comments and interest by the discusser.

The Author agrees that measurement of pore pressure build-up and dissipation is needed to better understand the liquefaction process in sand lenses. As we discussed in the paper, a miniature pore water pressure transducer was available, but was not used due to concerns of smearing or damage in the clay material. Although the idea of an electrical cable to measure pore pressure is good, it requires fixing the location of the sand lens, and provides a potential leakage route in the container. Since we plan to next study various sizes, shapes, arrangements, and numbers of lenses, we prefer the flexibility and portability of the miniature transducer. We expect to enclose the transducer in a permeable geotextile fabric that can be removed if smeared.

Unfortunately, the weight of the sand placed in each lens cavity was not measured, thus preventing the determination of unit weights and relative densities. However, this oversight will be corrected in future testing, in light of the important role that relative density plays in liquefaction of sand.

The Author agrees that partial pore water drainage could occur at the contact of the lens with the plexiglass wall. This condition could be eliminated by completely enclosing the sand lens in the clay sample, but this would prevent observation of deformation behavior and make the measurement of the deformed shape very difficult. We will try such an arrangement to determine the significance of partial pore water drainage on the liquefaction of sand lenses. We will also try to develop artesian pressure in a lens, since this can occur naturally and would make liquefaction easier to induce.