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SYNOPSIS: A laboratory based approach is described to obtain the post-liquefaction steady-state shear strength of very loose, recently deposited, layered fluvial silty sand deposits susceptible to lateral spreading. Triaxial specimens are formed by the Remolded Discontinuously Wet Pluvial Soil Sample (RDWPSS) preparation method. RDWPSS silty sand specimens are prepared by dumping the soil in water and allowing for segregation and development of layers. Ten undrained tests were conducted using ranges of consolidation pressures $\bar{\sigma}_{3c}$ = 0.2 to 2.7 kg/cm^2 and $K_c = \bar{\sigma}_{1c}/\bar{\sigma}_{3c}$ = 1 to 2. The experiments included both monotonic compression triaxial ($\overline{\text{CIU}}$) and cyclic torsional (CyT-$\overline{\text{CAU}}$) tests. The RDWPSS soil was contractive in all tests including those conducted at the lowest confining pressures. Unique steady-state lines are defined from the results of these tests. The ratio $S_{us}/\bar{\sigma}_{1c}$ is constant and equal to about 0.12 for K_c = 1 to 1.5, and it is proposed to use this value for lateral spread evaluations in conjunction with Newmark's sliding block analysis to predict permanent displacement. Finally, the approach is successfully used in a preliminary evaluation of the 18 cm lateral displacement measured at the Wildlife site in Southern California after the November 27, 1987 earthquake.

INTRODUCTION

Lateral spreading associated with the liquefaction of loose, saturated sand deposits constitute one of the most common and destructive phenomena caused by earthquakes. A lateral spread typically involves predominantly horizontal displacements of a large, superficial block of soil as a result of the liquefaction of a subsurface layer, with the displacements due to the combined effect of static and seismic forces. Lateral spreads generally develop downhill on very mildly sloping terrain containing late Holocene, loose, fluvially sedimented sand or silty sand. Permanent displacements ranging between a few centimeters and several meters have been observed, causing damage to roads, canals, embankments, buried pipes and foundations of buildings (NRC, 1985; Youd and Perkins, 1987).

Clearly, the evaluation of engineering effects of liquefaction at a site for a given earthquake implies the ability of predicting the magnitude and spatial distribution of permanent ground displacements. However, although we can currently predict if a site will liquefy or not with a reasonable degree of confidence using penetration charts, no such general method exists for the evaluation of the displacement. Intensive research efforts on the subject are currently underway, especially in the United States and Japan (Hamada, et al., 1986; US-Japan, 1988, 1989). The magnitude of the displacements depends on the intensity and duration of the ground shaking, and Youd and Perkins (1987) have developed empirical charts for the Western United States, valid for the worst site conditions, which give the maximum displacement as a function of earthquake magnitude and distance. No such charts are available for other site conditions. Under the assumption that displacements accumulate only during shaking, the use of Newmark's method (Newmark, 1965; Lambe and Whitman, 1969), in conjunction with a post-liquefaction steady-state or residual soil shear strength has been proposed. Castro (1987) backfigured a value of 100 psf for this strength at the Heber Road site in the Imperial Valley, CA, from the displacement observed after the 1979 earthquake.

A main problem in the application of the Newmark technique is the selection of the soil strength to use in the analysis (Baziar, et al., 1990). The use of laboratory techniques is complicated by the shallow depths and low confining pressures involved and the well-known problems associated with undisturbed sampling of granular soil under the water table. This paper proposes a laboratory approach for evaluating the *in situ* steady-state shear strength of very loose, water-sedimented silty sand, and applies it to a lateral spread case history.

THE RDWPSS SAMPLE PREPARATION METHOD

The proposed method determines the steady-state shear strength of the silty sand at large strains, S_{us}, from undrained monotonic and cyclic torsional tests. The definition of S_{us} is the same used by Castro (1987). In an attempt to simulate the geologic history and layered *in situ* structure of a loose, recent fluvial deposit, the specimens are formed by discontinuous sedimentation in water followed by consolidation and undrained testing. This Remolded Discontinuously Wet Pluvial Soil Sample (RDWPSS) preparation method is the same used by Vasquez-Herrera & Dobry (1989), and Vasquez-Herrera, et al. (1990) to successfully reanalyze the 1971 hydraulic fill flow failure in the San Fernando Dam. The same Batch SF7 sand sampled from the Lower San Fernando Dam, containing about 50% non-plastic silt under the #200 sieve, previously tested by Vasquez-Herrera and by GEI (1989), was used herein to determine S_{us} for the range of confining pressures relevant to lateral spread evaluations.

The RDWPSS preparation technique used at RPI is illustrated in Figure 1 for a 4-layer specimen. Boiled deaired water is poured into a 2" diameter, 4" high mold covered inside with a stretched traxial membrane, and fitted with an extension in the upper part of the molde (extension not shown in Fig. 1). Before dumping the first 1" soil layer, the water level is 1.5" above the bottom of the mold, and more water is poured later to have always the water level about 0.5" over the surface of the soil layer about to be dumped. Each 1" layer is formed by dumping an equal weight of soil (60 grams) and waiting enough time (30 minutes) for all soil particles to settle before pouring more water and dumping the next layer. The 30-minute time interval was based on calculations, and the authors later verified experimentally that more than 99% of the weight of the soil had sedimented at the end of the interval. A number of layers are deposited this way in the mold and extension, followed by removal of the extension and excess soil and water and the final, 4-layer configuration of the sample shown in Fig. 1.

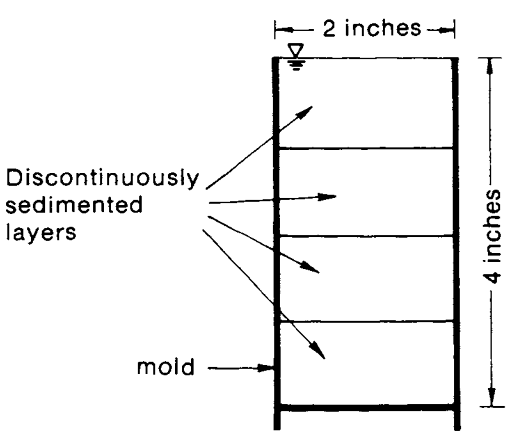

Fig. 1. Silty Sand Remolded Discontinuously Wet Pluvial Soil Sample (RDWPSS) Prepared Using Four Layers.

One of the tests was performed with a specimen having only one 4" thick layer so as to verify the influence of the number of layers on the results. In this case, the mold plus extension were filled with water to a height of 6", and the 240 grams of soil were dumped in the water at once. In both 4-layer and 1-layer specimens, the specimen was very loose after sedimentation, with an average void ratio, $e \cong 1.4$.

After sedimentation, the triaxial top cap and assembly was locked and an effective cell pressure of 3 psi (0.21 kg/cm^2) was applied by vacuum to the specimen which was then allowed to consolidate. Under this very low confining pressure, the soil consolidated significantly to an average void ratio, $e \cong 0.9$.

The RDWPSS method produces silty sand specimens that are non-uniform and have a naturally induced internal stratification, with an overall void ratio substantially larger than anything attainable by other preparation techniques aimed at creating more uniform specimens. It is reasonable to assume that the fabric, void ratio and engineering behavior of these RDWPSS specimens are similar to those of *in situ*, very loose, recently deposited, fluvial or other natural or man-made hydraulic fills. This was confirmed by Vasquez-Herrera, et al. (1990), who found that SF7 RDWPSS specimens consolidated to the field stresses had similar void ratios to those measured *in situ* in the Lower San Fernando Dam.

THE CyT-$\overline{\text{CAU}}$ TEST

Two types of undrained tests were utilized in this investigation. Isotropically consolidated, monotonic compression triaxial tests ($\overline{\text{CIU}}$) were used to obtain the steady-state strength S_{us} of the RDWPSS silty sand specimens. Anisotropically consolidated, cyclic torsional, strain-controlled tests (CyT-$\overline{\text{CAU}}$) were used to obtain S_{us} as well as the liquefaction flow failure triggering characteristics of the same soil. As shown by Vasquez-Herrera, et al. (1990) and verified later herein, the same values of S_{us}, as well as of other steady state characteristics, are obtained from both types of tests.

The CyT-$\overline{\text{CAU}}$ test, developed at RPI and previously described in detail by Vasquez-Herrera and Dobry (1989), is sketched in Fig. 2. The triaxial

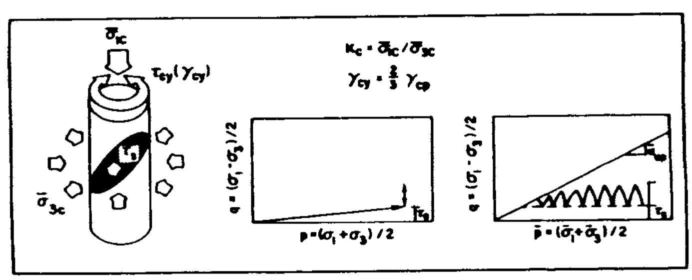

Fig. 2. Stress Conditions in CyT-$\overline{\text{CAU}}$ Test.

specimen is consolidated anisotropically to $\bar{\sigma}_{3c}$ and $\bar{\sigma}_{1c}$, with $K_c = \bar{\sigma}_{1c}/\bar{\sigma}_{3c} > 1$, and then is tested cyclically in an undrained condition by applying a torsional cyclic shear strain γ_{cy}. The representative value of γ_{cy} for the solid cylinder specimen is defined at two-thirds of the total radius of the specimen. After a number of strain cycles, n_t, liquefaction flow failure is triggered, large axial strains develop, the steady state is reached, and the shear strength drops to its steady-state value, S_{us}. In both $\overline{CIU}$ and CyT-$\overline{CAU}$ tests, the value of S_{us} is defined on the failure plane, $S_{us} = q_{us} \cos \bar{\phi}_{us}$, where both $q_{us} = (\sigma_{1us} - \sigma_{3us})$ and $\bar{\phi}_{us}$ are measured at steady state.

TEST PROGRAM

A total of ten tests were conducted on RDWPSS SF7 silty sand specimens, as listed in Table 1. Four tests were monotonic $\overline{CIU}$ experiments while the other six were cyclic CyT-$\overline{CAU}$ tests. In all tests, 4-layer specimens were used, with the exception of Test No. 4, which corresponds to a 1-layer specimen.

In the four CyT-$\overline{CAU}$ RDWPSS experiments on SF7 soil reported by Vasquez-Herrera and Dobry (1989), $\bar{\sigma}_{3c} \cong 1$ kg/cm^2 and $\bar{\sigma}_{1c} = 2$ kg/cm^2 to correspond to the conditions in the Lower San Fernando Dam. For

Test No.	Test Type	No. of Layers	$\bar{\sigma}_{3c}$ (kg/cm²)	$K_c = \bar{\sigma}_{1c}/\bar{\sigma}_{3c}$	γ_{cy}(%)
1	$\overline{CIU}$	4	0.91	1.00	---
2	$\overline{CIU}$	4	0.56	1.02	---
3	$\overline{CIU}$	4	0.21	1.05	---
4	$\overline{CIU}$	1	0.91	1.00	---
5	C_yT - $\overline{CAU}$	4	0.91	1.98	0.054
6	C_yT - $\overline{CAU}$	4	0.91	1.94	0.036
7	C_yT - $\overline{CAU}$	4	0.91	1.51	0.041
8	C_yT - $\overline{CAU}$	4	2.67	2.07	0.034
9	C_yT - $\overline{CAU}$	4	0.21	2.05	0.032
10	C_yT - $\overline{CAU}$	4	0.21	1.88	0.02

Table 1. Summary of Static and Cyclic Tests on SF7 RDWPSS Silty Sand.

the tests of Table 1, a much wider range of consolidation pressures was selected, $\bar{\sigma}_{3c} = 0.2$ to 2.7 kg/cm^2 and $\bar{\sigma}_{1c} = 0.2$ to 5.5 kg/cm^2; this range includes the low consolidation pressures of interest for evaluation of seismically-induced lateral spreads. Other parameters varied in the CyT-$\overline{CAU}$ tests were: K_c between 1 and 2, and γ_{cy} between 0.02% and 0.054%.

It is interesting to note that during consolidation, the ten RDWPSS specimens experienced additional significant reductions in their overall void ratio, with larger reductions occurring in the CyT-$\overline{CAU}$ tests. For the four $\overline{CIU}$ experiments listed in Table 1, the range was $e = 0.77$ to 0.90 (no difference was noted between the 1-layer and 4-layer specimens), while for the CyT-$\overline{CAU}$ tests, $e = 0.72$ to 0.81 after anisotropic consolidation.

TEST RESULTS

Figure 3 shows the results of $\overline{CIU}$ Test No. 1. The same as in the rest of $\overline{CIU}$ experiments, the stress-strain behavior is essentially elastic-perfectly plastic, with yield occurring at a strain of about 1%, and with a steady-state value of

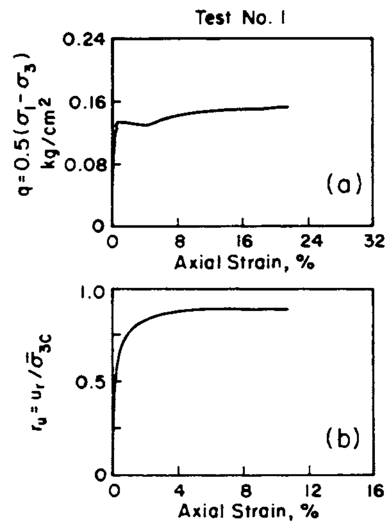

Fig. 3. Monotonic Undrained Triaxial Compression Test ($\overline{CIU}$).

$q_{us} = 0.15$ kg/cm^2 up to 22% strain. As will be shown later, $\bar{\phi}_{us} = 33.6°$, and thus $S_{us} = q_{us} \cos \bar{\phi}_{us} = 0.125$ kg/cm^2 for this specimen. Both the stress-strain and pore pressure ratio $r_u = u_r/\bar{\sigma}_{3c}$ curves show that the soil was contractive during the test. In fact, all $\overline{CIU}$ and CyT-$\overline{CAU}$ experiments listed in Table 1 exhibited contractive behavior, including those consolidated to the lowest confining pressure, $\bar{\sigma}_{3c} = 0.21$ kg/cm^2.

Figure 4 presents the results of CyT-$\overline{CAU}$ Test No. 6. After consolidating the specimen anisotropically to $\bar{\sigma}_{3c}$ and $\bar{\sigma}_{1c}$, the drainage valves were closed

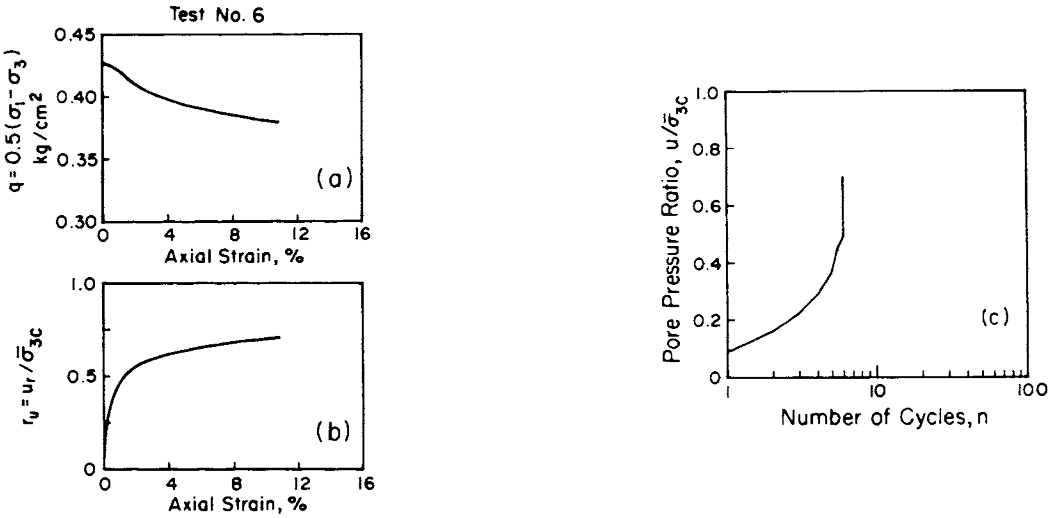

Fig. 4. Cyclic Torsional Test (CyT–$\overline{CAU}$).

and cyclic straining with γ_{cy} = 0.036% started. Flow failure triggering occurred at n_t = 6 cycles, when the accumulated axial strain was 0.8% and the pore pressure ratio was r_u = 0.43. Large axial strains of more than 10% and a further pore pressure increase developed during the seventh cycle approaching steady state. Values of q_{us} = 0.38 kg/cm^2 and S_{us} = 0.32 kg/cm^2 were obtained from this plot.

Figure 5 summarizes the steady-state and triggering characteristics of all tests listed in Table 1. Figures 5a and 5b include the $\bar{\sigma}_{3us}$ versus e and S_{us} versus e steady-state plots, while Fig. 5c

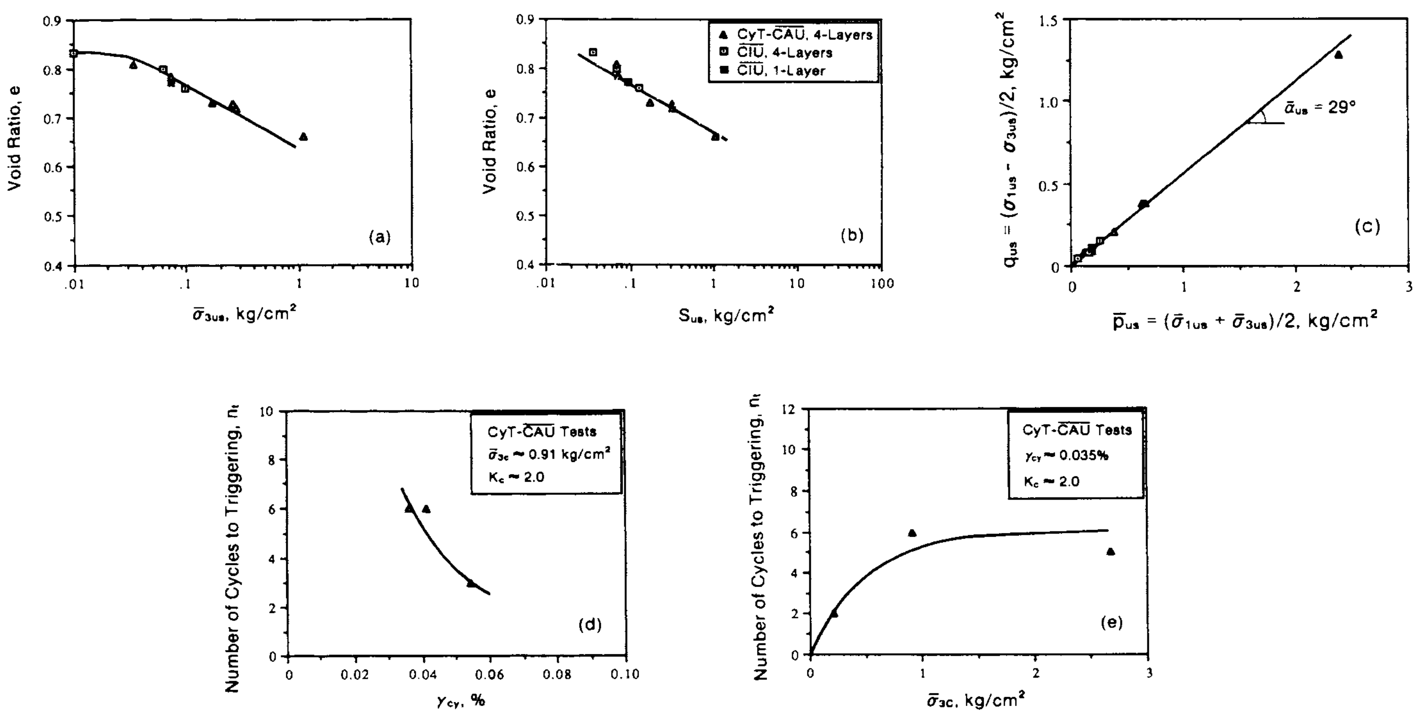

Fig. 5. Steady-State Strength and Triggering Characteristics from Monotonic and Cyclic Tests.

shows the strength envelope, giving $\bar{\alpha}_{us} = 29°$ and $\bar{\phi}_{us} = 33.6°$. These three plots clearly demonstrate that the steady-state characteristics of the RDWPSS soil are the same for monotonic and cyclic tests as well as for 1- and 4-layer specimens. The three plots are consistent with the location of the steady-state line for this soil obtained by Vasquez-Herrera and Dobry (1989) using only cyclic tests and a narrower range of confining pressures. Figures 5d and 5e plot n_t versus γ_{cy} and versus $\bar{\sigma}_{3c}$, respectively, using data furnished by the cyclic tests. The triggering curve of n_t versus γ_{cy} in Fig. 5d is similar to that used to reanalyze the San Fernando Dam flow slide, and it has the same general pattern found by Vasquez-Herrera and Dobry (1989) for this and other sands. The plot of n_t versus $\bar{\sigma}_{3c}$ in Fig. 5e extends the results in that publication to values of $\bar{\sigma}_{3c}$ larger than 1 kg/cm^2. It is interesting to note that although up to about 1 kg/cm^2 the curve could be approximated by a straight line as done by Vasquez-Herrera and Dobry, n_t stabilizes for larger values of $\bar{\sigma}_{3c}$.

$S_{us}/\bar{\sigma}_{1c}$ RATIO FOR LATERAL SPREAD EVALUATIONS

Steady-state plots of S_{us} versus e such as presented in Fig. 5b are typically used for flow failure evaluations of earth dams and slopes (Castro, 1987; GEI, 1985; Vasquez-Herrera & Dobry, 1989). In principle, the same approach could be used to provide the *in situ* S_{us} for analyzing lateral spreads. However, in either case, this requires an accurate knowledge of the *in situ* void ratio from precise *in situ* density measurements, which may not be available and can be difficult to acquire.

An alternative is to assume that the void ratio, fabric, and thus also the S_{us} of the RDWPSS remolded soil in the laboratory are the same as in the field for comparable consolidation stresses. This is reasonable for the recently deposited, very loose, water-deposited sediments which are mostly affected by liquefaction and lateral spreading. The authors (Vasquez-Herrera, et al., 1990) made a similar assumption and used the RDWPSS laboratory S_{us} in their successful reanalysis of flow failure at the San Fernando Dam.

The results of the four $\overline{\text{CIU}}$ tests are plotted in Fig. 6a as a graph of S_{us} versus $\bar{\sigma}_{1c}$. The figure indicates that the ratio $S_{us}/\bar{\sigma}_{1c}$ is constant and equal to about 0.12. This is a very important conclusion which allows estimating S_{us} in the field without the need for *in situ* density measurements.

A similar "c/p ratio" approach has been used for many years to estimate the undrained peak shear strength of cohesive soil deposits from laboratory tests, and was further developed in the last 15 years in their SHANSEP method by Ladd and Foott (1974). They relate this ratio to the over-consolidation ratio of the clay, OCR, with the smallest value corresponding to normally consolidated soil, and with the "c/p ratio" increasing as OCR increase. In the case of interest

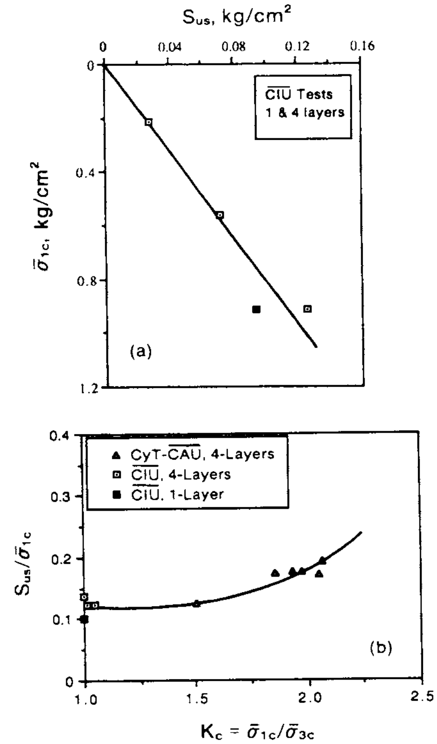

Fig. 6. $S_{us}/\bar{\sigma}_{1c}$ from Monotonic and Cyclic Tests.

here of layered silty sandy soil susceptible to liquefaction, the "$S_{us}/\bar{\sigma}_{1c}$ ratio," where $\bar{\sigma}_{1c} = \bar{\sigma}_v$ can be taken as the *in situ* vertical effective pressure, should be related to the density of the soil at deposition time (which in turn is controlled by factors such as the velocity of the water) and to subsequent changes throughout the life of the deposit (e.g., due to overconsolidation, densification caused by vibrations, etc.). Here the assumption is made that the value $S_{us}/\bar{\sigma}_{1c} = S_{us}/\bar{\sigma}_v \cong 0.12$ from Fig. 6a is directly applicable to very loose, normally consolidated, recently deposited soil.

Figure 6b shows $S_{us}/\bar{\sigma}_{1c}$ as a function of K_c from the results of the ten $\overline{\text{CIU}}$ and CyT-$\overline{\text{CAU}}$ tests of Table 1. The ratio is about constant and approximately equal to 0.12 for K_c between 1 and 1.5, but increases rapidly for $K_c > 1.5$, with $S_{us}/\bar{\sigma}_{1c} \cong 0.18$ for $K_c = 2$. This graph can be useful to estimate S_{us} of soil in a slope or beneath an embankment where $K_c > 1$ and a driving shear stress is present during undrained loading.

APPLICATION

The $S_{us}/\bar{\sigma}_v = 0.12$ ratio from Fig. 6 was used for a preliminary evaluation of the lateral displacement developed at the Wildlife site in Imperial Valley, Southern California, by the 11/27/87 Superstition Hills earthquake.

During this 6.6 magnitude event, the site liquefied, developed sand boils and cracks and moved horizontally up to 230 mm toward the Alamo River. The site had been instrumented with both accelerometers and piezometers. For the first time, pore pressure ratios of 100% were measured in the field in a saturated sandy deposit during an earthquake. Also, field exploration conducted at this site provided useful information on the soil profile (Youd & Bartlett, 1988; Holzer, et al., 1989; Dobry, et al., 1990).

Figure 7 shows the N15E cross-section of interest and the soil profile. The site is mostly flat with an essentially vertical free face at the river's edge. Layer A, between 0 and 2.5 m depth, consists of very loose and very soft interbedded micaceous sandy silt, silt, and clayey silt (N = 1 to 3 blows/ft). It is followed by a silty sand layer B between 2.5 m and 6.8 m, underlain by a stiff to medium stiff clayey silt. Silty sand layer B, in turn, is divided into two sublayers: very loose to loose sublayer B_1 between 2.5 m and 3.5 m, with small-scale cross-bedding ($N \cong 5$ blows/ft), and loose to medium dense sublayer B_2 between 3.5 m and 6.8 m (N = 6 to 13 blows/ft). The groundwater level is at 1.5 m depth. Accelerometers recorded the earthquake at the ground surface, and at 7.5 m depth within the stiff to medium clayey silt underlying the liquefiable layers A and B. Several piezometers in layer B recorded up to a 100% pore pressure ratio in this earthquake.

Measurements showed that parts of the site had moved laterally by various amounts. Cracks opened parallel to the river at about 17.7 m from the free face, and a 18 cm lateral movement was measured in an approximately N15E direction towards the river (Fig. 7). Based on the soil profile of Fig. 7 and some analyses, Dobry, et al. (1989) suggested that yielding and lateral straining associated with the liquefaction of loose layers A and B_1 were responsible for this lateral movement. Holzer, et al. (1989) independently arrived at a similar conclusion based on the post–earthquake monitoring of an inclinometer located near the cracks in Fig. 7, which indicated that a large subsurface shear strain had occurred in sublayer B_1.

Based on the information above, the authors postulated the two failure mechanisms sketched in Fig. 7. Both failure planes QR and QR' start at point Q defined by the intersection of the cracks and the groundwater level, and they end at the free face at the base of layers A and B_1, respectively. These failure planes define angles $\alpha = 3.23°$ and $\alpha = 6.43°$, used in the Newmark sliding block analyses (Newmark, 1965) to evaluate the lateral movement toward the river of rigid block PQRS or PQR'S.

For a very mildly sloping failure surface as is the case here, and assuming that the earthquake excitation is parallel to the failure plane, it can be shown that the yield acceleration, a_y, needed to start the block sliding toward the river is given by:

$$a_y/g = (\bar{\sigma}_v/\sigma_v)\ (\cos\alpha \tan\phi - \sin\alpha) \tag{1}$$

where g = acceleration of gravity, and ϕ is an "equivalent angle of internal friction" related to the ratio $S_{us}/\bar{\sigma}_v$ by the expression:

$$(S_{us}/\bar{\sigma}_v) = (1 - \sin\phi)\tan\phi \tag{2}$$

Note that this ϕ is related to the ratio between S_{us} and the consolidation stress $\bar{\sigma}_v = \bar{\sigma}_{1c}$, and

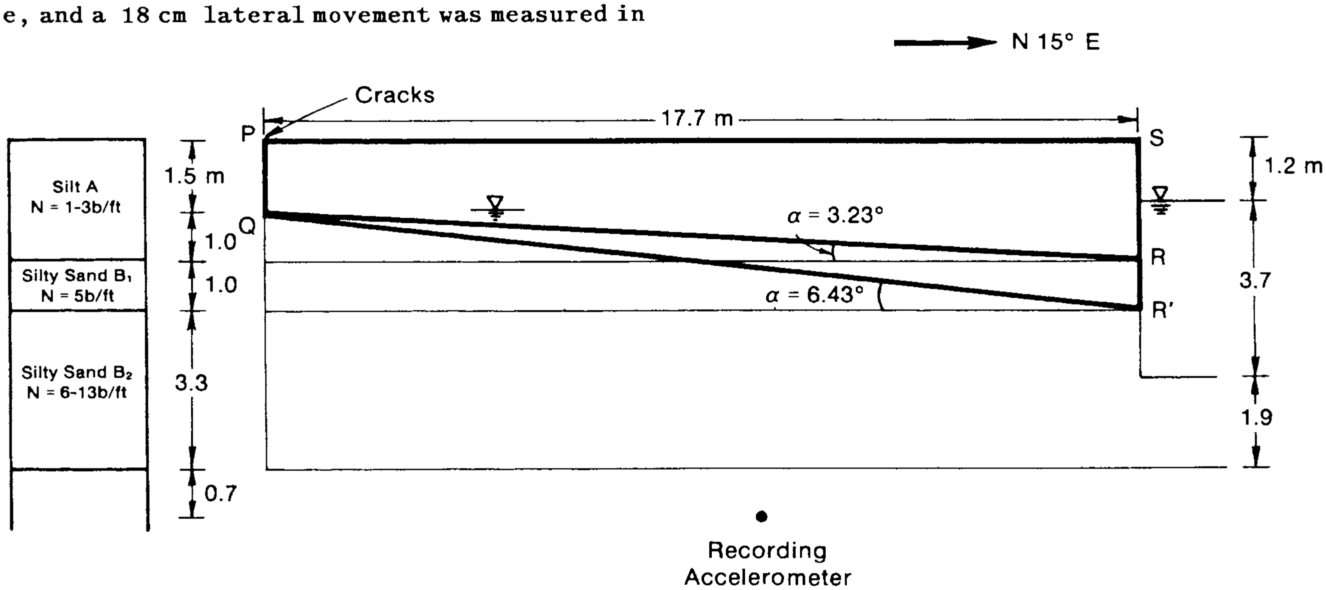

Fig. 7. Soil Profile and Assumed Failure Planes for Newmark Analyses, Wildlife Site, November 24, 1987 Earthquake.

therefore has no relation to $\overline{\phi}_{us}$ in Fig. 5c. For $S_{us}/\overline{\sigma}_v = 0.12$, $\phi = 8°$. It can also be demonstrated that for a case such as shown in Fig. 7, $\overline{\sigma}_v$ and σ_v can be taken at the midpoint of failure plane QR or QR'. Finally, $a_y = 0.074$ g and $a_y = 0.022$ g are found for QR and QR', respectively.

Figure 8a includes the NS component of the accelerogram recorded at 7.5 m depth, below the liquefied soil. Based on the recorded ground surface accelerogram and the site response analyses reported by Dobry, et al. (1989), the ordinates of this record were multiplied by a factor 1.05 prior to using it in the analyses, to account for the amplification of the motions between 7.5 m depth and the base of the sliding block, at 2 of 3 m depth.

Figure 8b presents the displacement time histories calculated with the sliding block analyses for the two values of α. In the analyses, the block was only allowed to slide toward the river, on the assumption that the material filling up the cracks did not permit a sliding back up of the block away from the free face. For $\alpha = 3.23°$ (block PQRS), a total displacement of 0.21 cm is calculated at the end of the earthquake, while for $\alpha = 6.43°$ (block PQR'S), the calculated displacement is 40.5 cm. These two values bound the measured displacement of 18 cm, as illustrated by the figure, thus showing the reasonableness of the assumed failure mechanism and value of $S_{us}/\overline{\sigma}_v$ used in the calculations. The wide variation between these two calculated values of displacement illustrates the sensitivity of the prediction to the selected angle α.

CONCLUSIONS

The use of the RDWPSS laboratory technique to simulate very loose fluvial deposition of silty sand in the laboratory is clearly demonstrated. The method produces contractive specimens even at very low confining pressures which have the layered, segregated fabric of the in situ soil. Furthermore, the same steady-state characteristics are obtained for a wide range of confining pressures independently of the character of the test (monotonic or cyclic), type of consolidation (isotropic or anisotropic) and number of layers (one or four).

The use of a constant ratio $S_{us}/\overline{\sigma}_{1c}$ as measured in the tests presented here offers a promising way of defining the in situ steady-state or residual strength, without the need to know accurately the density of the soil in the field. The yield acceleration needed for a sliding block analysis of a lateral spread is a simple function of $S_{us}/\overline{\sigma}_{1c}$, the state of total and effective vertical pressures, and the geometry of the sliding block.

Further confirmation of the reasonableness of the approach is provided by the successful preliminary evaluation of the lateral displacement measured at the Wildlife site after the November 1987 earthquake.

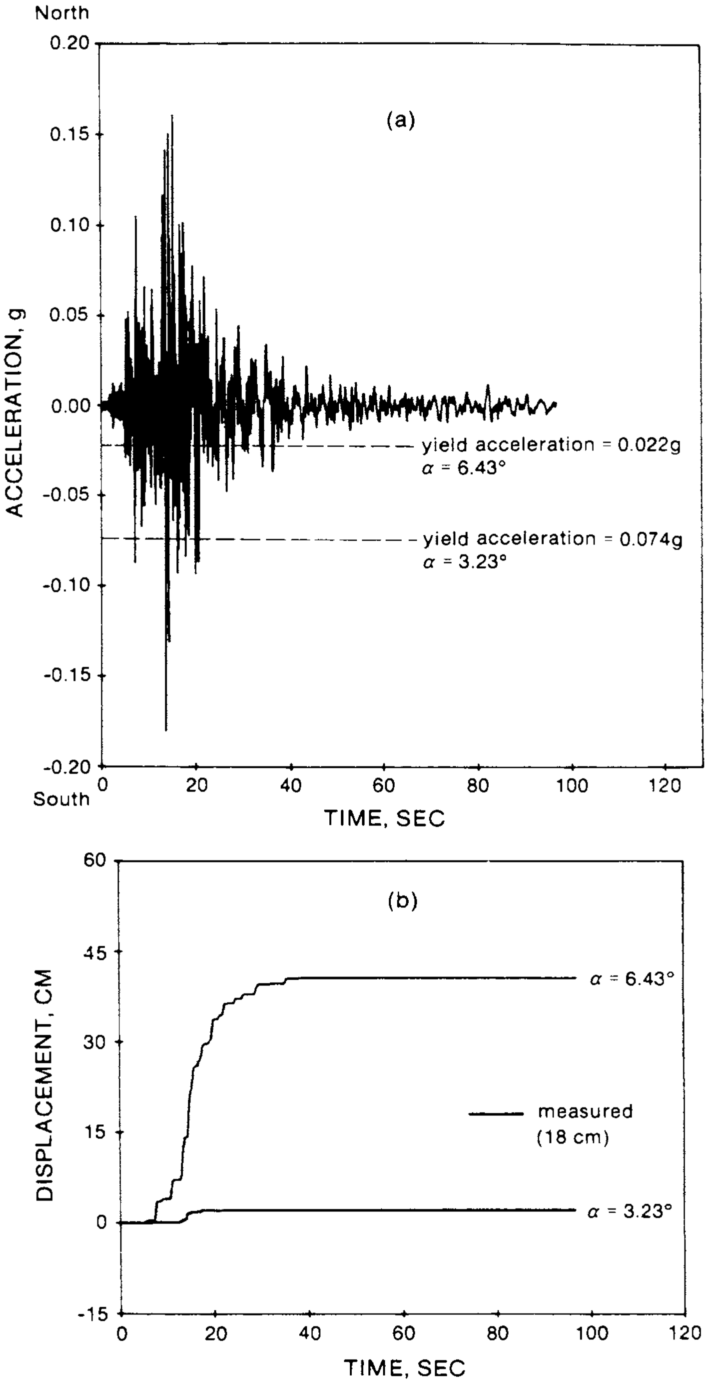

Fig. 8. Measured and Predicted Displacement at Wildlife Site Using Newmark Analysis, November 24, 1987 Earthquake.

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