Steady-State Strength Analysis of Lower San Fernando Dam Slide

By Gonzalo Castro,[1] Raymond B. Seed,[2] Thomas O. Keller,[3] Members, ASCE, and H. Bolton Seed,[4] Fellow, ASCE

Abstract: An extensive reevaluation of the seismically induced 1971 Lower San Fernando dam slide is performed using steady-state strength concepts. This investigation includes both field and laboratory testing and is carried out as a cooperative effort by four organizations. The undrained steady-state strength (S_{us}) of the hydraulic fill is evaluated based on laboratory testing of undisturbed samples obtained (1) From boreholes; and (2) by hand trimming in a large-diameter exploratory shaft. The resulting S_{us} values are corrected for the effects of void-ratio changes due to sampling and testing, as well as for void-ratio changes due to the 1971 earthquake shaking. The strength of the hydraulic fill is also backfigured from the extent of the slide movements and compared with the measured (corrected) strengths; the results indicate that the strength mobilized during the failure is about equal to the average minus about one half to the average minus one full standard deviation values of the measured (corrected) laboratory strength estimates.

Introduction

The Lower San Fernando Dam in Southern California developed a major slide in the upstream slope and crest as a result of the 1971 San Fernando earthquake. The slide occurred due to liquefaction (loss in strength) of a zone of hydraulic sand fill near the base of the upstream shell. An investigation of the slide, including trenches and borings, in situ density tests, undisturbed sampling, index testing, static and cyclic load testing, and analyses, was performed and reported by Seed at al. (1973, 1975a) and Lee et al. (1975).

Two cross sections of the Lower San Fernando Dam are presented in Fig. 1, one showing the observations made in a trench excavated through the

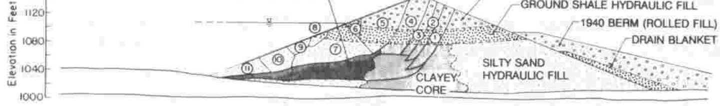

FIG. 1. Cross Section through Lower San Fernando Dam Showing: (*a*) Conditions after 1971 Earthquake; and (*b*) Schematic Reconstruction of Failed Cross Section

[1]Prin., GEI Consultants, Inc., Winchester, MA 01890.
[2]Prof., Dept. of Civ. Engrg., Univ. of California, Berkeley, CA 94720.
[3]Proj. Mgr., GEI Consultants, Inc., Winchester, MA 01890.
[4]Formerly, Cahill Prof. of Civ. Engrg., Univ. of California, Berkeley, CA 94720.

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1971 slide area, and the other showing a reconstructed cross section of the dam, illustrating the zone in which liquefaction occurred. Large blocks of essentially intact soil from the upstream section of the dam moved into the reservoir, riding over or "floating" on the liquefied soil. After movements stopped, the liquefied soil was found to have extruded out beyond the toe of the dam and up between the intact blocks, with maximum movements of as much as 200 ft (61 m) beyond the toe of the dam. The block of soil that contained the toe of the dam moved about 150 ft (46 m) into the reservoir.

Data from seismoscopes, located on the abutment and on the crest of the embankment, indicated peak accelerations of about 0.55 g and 0.5 g, respectively, and an analysis of the seismoscope record on the dam crest indicated that the major slide started about 40 sec after the earthquake shaking had stopped (Seed 1979). Thus, the large slide movements apparently developed in the absence of earthquake-induced stresses and were caused by the static stresses due to the weight of the materials in the embankment. It can thus be inferred that the earthquake shaking triggered a loss of strength in the soils comprising the embankment, and it was this loss of strength, rather than the inertia forces induced by the earthquake shaking, that led to the sliding of the upstream slope.

This paper presents the results of laboratory and field investigations in which the "steady-state" analysis procedures proposed by Poulos et al. (1985) were applied to a reanalysis of the postearthquake stability of the Lower San Fernando Dam. These steady-state analysis procedures involve the use of laboratory tests of high-quality "undisturbed" samples, together with corrections for inevitable sample disturbance, to estimate the post-liquefaction residual undrained strength or steady-state strength of in situ soils. These testing and analysis procedures were applied to the Lower San Fernando Dam, and the resulting estimated in situ steady-state strength was compared with the actual field value as backcalculated from the observed failure. The results of these studies thus provide valuable insight regarding the validity and reliability of these laboratory-based steady-state testing procedures.

Laboratory-Based Steady-State Methodology

During the slide in the Lower San Fernando Dam, the liquefied soil underwent extremely large deformations. These deformations were not cyclic, but essentially unidirectional, i.e., downslope and toward the reservoir. The earthquake-induced cyclic strains and associated stresses triggered the initial loss of strength in the hydraulic fill that led to the failure, but the response of the soil in liquefying and flowing was a result of its low undrained strength under large unidirectional deformations. When a soil is strained unidirectionally in an undrained condition, at some point, it reaches a steady state of deformation (Poulos et al. 1985) in which the shear resistance becomes constant and is the lowest value that a contractive mass of soil can have at its particular void ratio. The minimum strength is termed the undrained steady-state strength, S_{us}, and is primarily a function of the density (or void ratio) of the soil (Castro et al. 1982).

The steady state of deformation for any soil mass deforming unidirectionally is defined as that state in which the mass is deforming continuously at constant volume, constant effective normal stress, constant shear stress, and constant velocity. The steady state of deformation is achieved only after all particle orientation has reached a statistically steady-state condition and after all particle breakage, if any, is complete, so that the shear stress needed to continue deformation remains constant. The steady state of deformation is the same concept as that envisioned by Casagrande (1936) when he proposed the existence of a critical void ratio, or critical density, for sands.

Poulos et al. (1985) have proposed a methodology for the evaluation of in situ steady-state strength based on careful laboratory testing of high-quality undisturbed samples. It is described in detail in Poulos et al. (1985) and illustrated schematically in Fig. 2. Basically it recognizes that samples of loose-to-medium-dense sands and nonplastic sandy silts are likely to be densified in the sampling, transportation, handling, and testing procedures. Thus the laboratory steady-state strength of the soil $(S_{us})_L$ is measured at the void ratio at the time of failure in the laboratory (e_L). Previous investigations have shown that for a given soil: (1) The slope of the steady-state line on a semilog plot is affected chiefly by the shape of the grains; and (2) the position of the steady-state line is influenced chiefly by the soil's grain-size distribution (Castro et al. 1982). Accordingly, the steady-state strength measured in the laboratory is corrected to a lower value $(S_{us})_f$ corresponding to the void ratio of the soil in its field condition (e_f) by assuming that the slope of the steady-state line (the relationship between steady-state strength and void ratio) is the same for undisturbed and remolded samples. The resulting (corrected) values of $(S_{us})_f$ are a sensitive function of the in situ void ratio of the specimens tested in the laboratory, and of the slope of the steady-state line determined for remolded samples. Accordingly, associated with the development of this procedure has been the development of improved procedures for obtaining undisturbed samples of sand for laboratory testing purposes, and techniques for accurately measuring void ratio changes during sampling, transportation, handling, and testing.

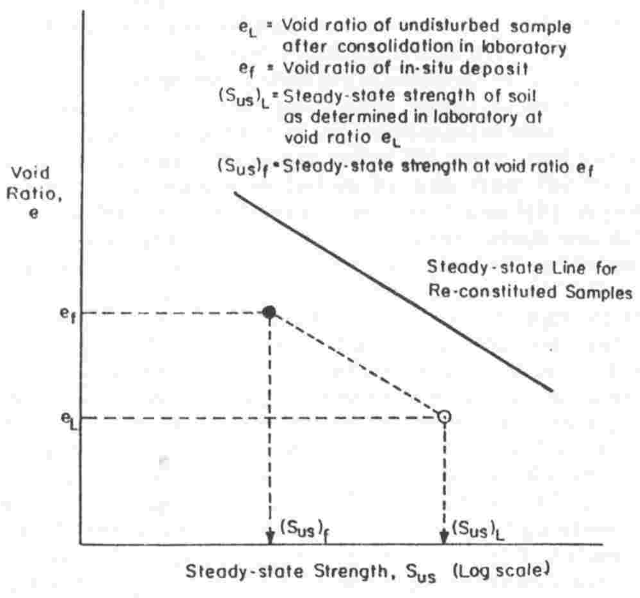

FIG. 2. Procedure for Determining Steady-State Strength of Soil at Field Void Ratio (Poulos et al. 1985)

Lower San Fernando Dam

Figure 1(*b*) shows a typical cross section through the Lower San Fernando Dam as it existed immediately prior to the 1971 earthquake. Embankment construction began in 1912. The embankment was founded on an alluvium foundation consisting primarily of stiff clay with layers and lenses of sand and gravel.

The majority of the embankment consists of hydraulic fill placed between 1912 and 1915. This material was sluiced from the floor of the reservoir and discharged from starter dikes on the upstream and downstream edges of the embankment. The actual dimensions of these starter dikes are unknown. The hydraulic-fill process resulted in upstream and downstream shells consisting primarily of sands and silts and a central core consisting primarily of clayey soils. Construction photos of the hydraulic-fill placement and past reports indicate that the upstream and downstream sections were raised symmetrically and constructed in a similar manner. Therefore, it is reasonable to assume that the general characteristics of the upstream and downstream hydraulic fill shells are similar.

A 10- to 15-ft-thick hydraulic fill layer consisting of ground-up shale from the left abutment was placed in 1916 over the initial hydraulic fill described previously. Limited sampling of the ground shale in 1985 disclosed a widely graded sand and silty sand, and construction records indicate that the maximum particle size of the ground shale was about 3 in.

The embankment was raised a number of times between 1916 and 1930 by placement of rolled fills. The maximum height of the embankment of about 140 ft was reached in 1930. A thin blanket was placed on the lower part of the downstream slope in 1929 and 1930, apparently for seepage control and to provide additional stability due to the raising of the crest. The composition of the blanket was described in a postconstruction report as a mixture of shale and gravelly material placed in 12-in. layers and compacted by trucks.

The final addition to the dam was a 4.5H:1V berm placed on the downstream slope in 1940. Construction records related to the composition of the berm could not be found, but it has been described in a previous report (Baumann et al. 1966) as a rolled fill. A photograph of the construction operation shows a roller traveling on the fill.

Steady-State Strength Backfigured from Slide

It is possible to estimate the value of the undrained steady-state strength (S_{us}) at the base of the upstream hydraulic fill zone through which the failure occurred by analyzing the movements of the observed failure mass. These analyses are complicated to some extent by uncertainties associated with the complexity of the actual failure mechanism and by the need to account for dynamic (momentum) effects.

Theoretically, an upper bound for the value of S_{us} can be obtained by means of a stability analysis of the prefailure configuration, as illustrated schematically in Fig. 3(*a*). Analyses performed using different assumptions indicate average driving shear stress values ranging from 850 to 1,050 psf, i.e., a factor of safety of one is obtained in the stability analysis when one assumes these strength values in the lower zone of the hydraulic fill shell. It is clear, however, that the actual field strength must have been considerably lower than this. If S_{us} within the upstream hydraulic fill had indeed been close to 850 psf, then small slide movements, on the order of 10 to 15

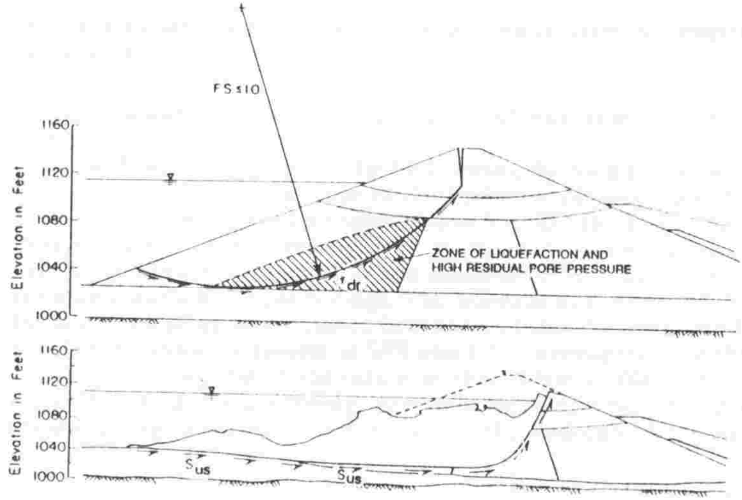

FIG. 3. Backcalculation of Initial and Final Shear Stresses within Lower San Fernando Dam: (*a*) Conditions at End of Earthquake, Immediately Prior to Sliding; and (*b*) Postfailure Configuration at End of Slide Movements

ft or so, would have been sufficient to reduce the static driving forces and would have brought the slide to a halt. The fact that very large movements, on the order of 150 ft, occurred before sliding stopped indicates that the undrained steady-state strength of the liquefied hydraulic fill zone was considerably less than 850 psf.

Similarly, a theoretical lower-bound estimate of the undrained residual strength ($\bar{S}_{us}$) of the soil at the base of the upstream hydraulic fill zone can be obtained by considering the postfailure configuration, as shown in Fig. 3(*b*). At this postfailure condition, potentially higher-strength soils associated with the starter dikes at the toe of the embankment have been sheared completely through, as have the upper rolled fill zones, and virtually the entire base of the slide mass is underlain by soils from the base of the upstream hydraulic fill zone, as illustrated in Figs. 1 and 3(*b*). A portion of the slide plane passes through the clayey core zone, which was estimated to have an undrained steady-state strength of $S_{us} \approx 600$ psf based on investigations reported by Seed et al. (1973) and Castro and Keller (1988). There is significant uncertainty regarding the shear strength of the soils underlying the portion of the slide mass that entered the reservoir (passing beyond the initial upstream toe of the embankment), because of the possibility that the sliding mass entering the reservoir may have entrapped some water and/or loose sediments on the base of the reservoir. Assumptions for the shear strength under the portion of the slide mass that entered the reservoir, ranging from 0% to 100% of the S_{us} within the embankment hydraulic fill, lead to computed S_{us} values for the hydraulic fill ranging from approximately 150 psf to 250 psf for the postfailure configuration.

The aforementioned computed upper- and lower-bound $\bar{S}_{us}$ values of about 850 psf and 200 psf, respectively, correspond to equilibrium under assumed static (zero acceleration) conditions for the pre- and postfailure configurations. A more accurate and reliable assessment of the actual S_{us} of the soils at the base of the upstream hydraulic fill zone requires consideration of dynamic effects. The various writers of this paper have different opinions regarding how this can best be accomplished, and differing opinions regarding the values of S_{us} to be assigned to the base of the portion of the slide mass that entered the reservoir. Using a variety of techniques to account for dynamic or momentum effects, and a range of assumed S_{us} values for the base of the slide mass within the reservoir, estimated values of the actual field undrained steady-state strength of the soils at the base of the upstream hydraulic fill zone range from $S_{us} \approx 300$ psf to $S_{us} \approx 550$ psf (Castro and Keller 1988; Davis et al. 1988; Seed et al. 1988). Considering the uncertainties involved, the authors have agreed to a consensus opinion that the best estimates of the actual field undrained steady-state strength are in the range of $S_{us} \approx 400$ psf to 500 psf. Extensive analyses and discussions on this issue clearly demonstrated that there are significant uncertainties involved in backfiguring S_{us} values from the slide, despite the unusually large body of information available regarding this slide event.

1985 Field Investigation and Sampling

An extensive study, including in situ testing as well as sampling and laboratory testing, was performed during the period 1985–1987 with the main purpose of determining S_{us} values in the hydraulic fill shell using the methodologies proposed by Poulos et al. (1985). The detailed results of the investigation are reported by Seed et al. (1988) and Castro et al. (1988).

Since the failure of the upstream slope of the Lower San Fernando Dam in 1971, the dam has been reconstructed to serve as an emergency water-retaining structure with the configuration shown in Fig. 4. The original upstream shell has been replaced by a compacted fill, but the downstream shell below El. + 1,100 remains essentially as it was at the time of the 1971 earthquake. Since the original hydraulic-fill embankment was probably symmetrical in configuration and properties about the centerline of the crest, the properties of the soil forming the upstream shell can be evaluated with a reasonable degree of accuracy on the basis of the properties of the hydraulic fill comprising the present downstream portion of the embankment.

An exploration program was carried out in 1985 consisting of the following:

1. Six standard penetration test (SPT) borings and 12 cone penetration test (CPT) soundings along four cross sections to define the character of the materials in the dam.
2. Undisturbed sample borings adjacent to five selected SPT/CPT locations.
3. One deep exploration shaft located adjacent to an SPT/CPT location to

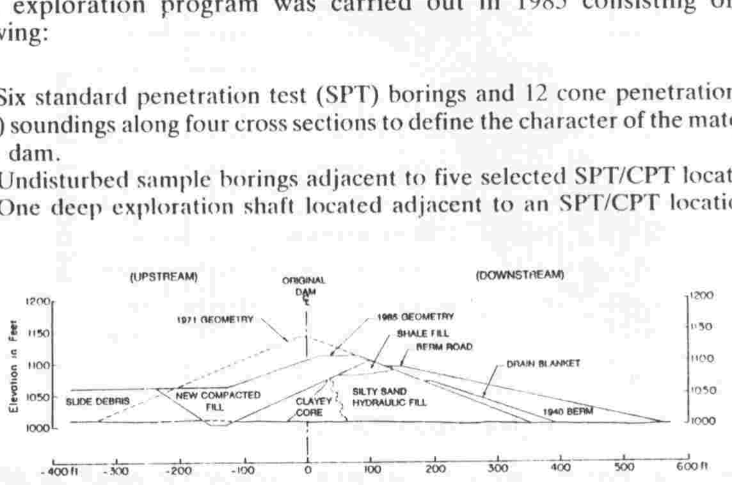

FIG. 4. Current Configuration of Reconstructed Lower San Fernando Dam

obtain undisturbed samples, perform in situ density tests, and map the sidewalls of the shaft.

The locations of the various field tests and borings are shown in plan view in Fig. 5.

In the field investigations, the SPT boring showing the most consistently low blow counts near the base of the hydraulic fill was found to be boring S111 on the cross section through station 5 + 85. The exploration shaft was thus excavated near to boring S111 in order to obtain high-quality (hand-carved) undisturbed samples of this material in addition to those obtained from undisturbed sample borings. The material at the base of the hydraulic fill at this location was found to be a layer of stratified silty sand and sandy silt. Fig. 6 shows the results of SPT and CPT investigations at this location.

In interpreting the stratification in the hydraulic fill, five major zones were identified in each boring, and these were designated as zones 1–5 as shown in Fig. 6. Zone 5, at the base of the hydraulic fill, corresponds to

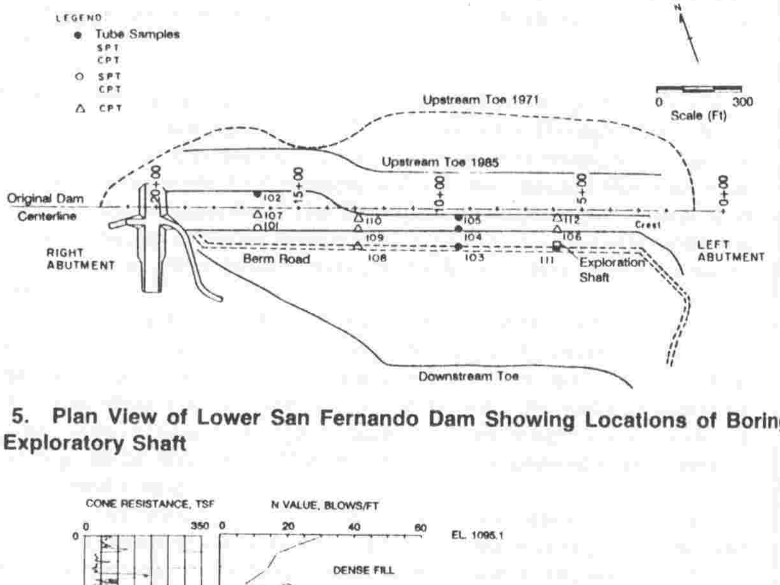

FIG. 5. Plan View of Lower San Fernando Dam Showing Locations of Borings and Exploratory Shaft

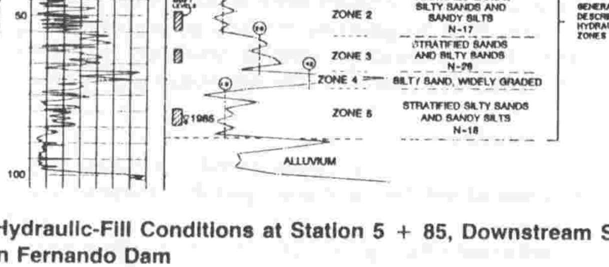

FIG. 6. Hydraulic-Fill Conditions at Station 5 + 85, Downstream Shell, in 1985; Lower San Fernando Dam

the mirror image of the zone through which the upstream slide mainly took place, and, thus, most of the sampling and testing was concentrated in this zone. It is to be expected that within zone 5, the gradation of the soils will change gradually from cleaner sands near the starter dike to siltier sands and silts near the core. Most of the sampling of this zone was made from borings and from the shaft located on the road at the top of the downstream berm. As shown in Figs. 1, 4, and 5, this location corresponds to about the midpoint of the mirror image of the failure zone in the upstream shell. Thus, the sampling probably corresponds reasonably well to the average soil conditions in the weakest part of the liquefied zone.

Zone 5 is approximately 15 ft thick and consists primarily of stratified silty sands and sandy silts. Fig. 7 shows the gradation curves of undisturbed samples retrieved from zone 5. The hydraulic fill soils within this zone were highly stratified, with roughly 20 macro layers, each 9–12 in. (22–30 cm) thick. Micro layers on the order of 1 mm thick are also extensive throughout this zone. Standard Penetration Test blow counts within zone 5, from both the 1971 and 1985 field investigations, corrected for overburden effects as well as equipment and procedural effects according to Seed et al. (1984), ranged from $(N_1)_{60}$ = 6 blows/ft to 28 blows/ft, with an average value of $(N_1)_{60} \approx 13$ blows/ft (Seed et al. 1988).

Laboratory-Based Evaluation of S_{us}

The determination of the undrained steady-state strength by means of laboratory tests involves the following steps, as illustrated in Fig. 2.

1. Determine the in situ void ratio (prefailure), e_f.

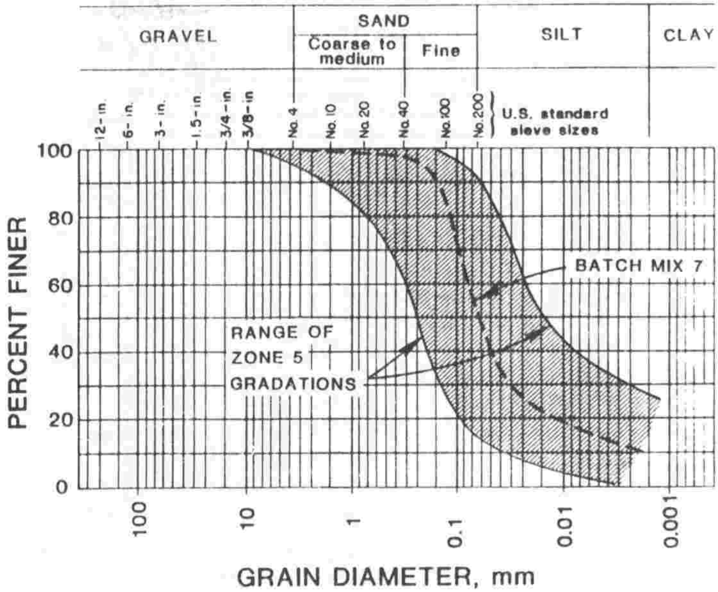

FIG. 7. Range of Gradations of Hydraulic-Fill Samples from Zone 5 at Base of Lower San Fernando Dam

2. Determine the slope of the steady-state line from a plot of void-ratio versus undrained steady-state strength from tests on reconstituted samples.

3. Determine the undrained steady-state strength for undisturbed specimens, $(S_{us})_L$.

4. Correct the laboratory-measured undrained steady-state strengths from step 3 to S_{us} values representative of the in situ (field) void ratio, $(S_{us})_f$.

The following sections describe in detail these steps.

Determination of In Situ Void Ratios

A key to the proper determination of undrained steady-state strength is the determination of the in situ void ratio of the soil prior to the 1971 slide. The discussion that follows presents the measurements of in situ void ratios in 1985 and estimates of the changes that took place since the time of the earthquake.

In situ void ratios of the critical hydraulic fill layer (zone 5) on the downstream side of the dam were determined using three methods: fixed-piston sampling in boreholes, tripod tube sampling, and field density testing in the exploration shaft.

- Fixed-piston sampling—Undisturbed samples of the critical layer were obtained from borings using a 7.1-cm-diameter Hvorslev-type fixed-piston sampler. Careful measurements of sampler penetration and soil recovery were made to document soil volume changes that may have occurred during sampling. The fixed-piston sampling procedures used are described in detail by Poulos et al. (1985) and Keller (Geotechnical Engineers Inc., Winchester, mass., unpublished internal memorandum, 1981).
- Tripod tube sampling—Undisturbed samples of the critical layer were obtained from the floor of the large-diameter exploration shaft using the GEI tripod sampler, as described by Marcuson and Franklin (1980). The procedure involves advancing a tube into the soil in increments using hand-carving techniques, such that any length changes during carving can be measured. The sampling tube alignment is kept vertical by a tripod frame.
- Field density testing—Void ratios of the critical layer were also determined by field density tests using the sand cone technique. These tests were performed at the floor of the exploration shaft, adjacent to the tripod tube sampling locations.

Appropriate corrections were made to the measured void ratios to obtain 1985 in situ void ratios. Corrections were made for the volume changes that occurred during tube sampling and for swelling of soils prior to sampling caused by unloading at the base of the exploration shaft. The following types of corrections were made for the various void-ratio determination methods:

- Fixed-piston samples—during sampling.
- Tripod tube samples—during sampling and due to swell, during shaft excavation.
- Field density tests—due to swell, during shaft excavation.

The lengths of fixed-piston and tripod tube samples were measured prior to extrusion in the laboratory. No changes in length of these samples took place during transportation or tube cutting, and therefore, no void-ratio corrections were necessary for these effects.

A plot of 1985 in situ void ratios of the hydraulic fill soils within the critical layer versus elevation is presented in Fig. 8. A unique relationship between in situ void ratio and depth would not be expected because of the gradation differences among the samples (Fig. 7). It is interesting to note that there is no consistent systematic difference between the corrected in situ void ratios obtained from fixed-piston samples, tripod tube samples, or field density tests. Void-ratio measurements of the critical layer were made at two locations on the downstream slope. These are locations 111 and 103, about 350 ft apart, as shown in Fig. 5. The data indicate that void ratios of the critical layer are relatively consistent across substantial distances parallel with the dam axis.

In order to evaluate the behavior of the Lower San Fernando Dam during the earthquake in 1971, it is necessary to determine the properties of the hydraulic fill for the conditions at the time the earthquake occurred. For some properties, such as drained strength, any volume changes since the earthquake may be of minor significance. However, the undrained steady-state strength is very sensitive to small changes in void ratio, and, thus, it is important to perform an accurate evaluation of the void ratio of the soil in its preearthquake condition.

Estimates of the changes in dry density or void ratio of the hydraulic fill since the time of the earthquake were made based on comprehensive settlement and deflection measurements of the face of the downstream shell

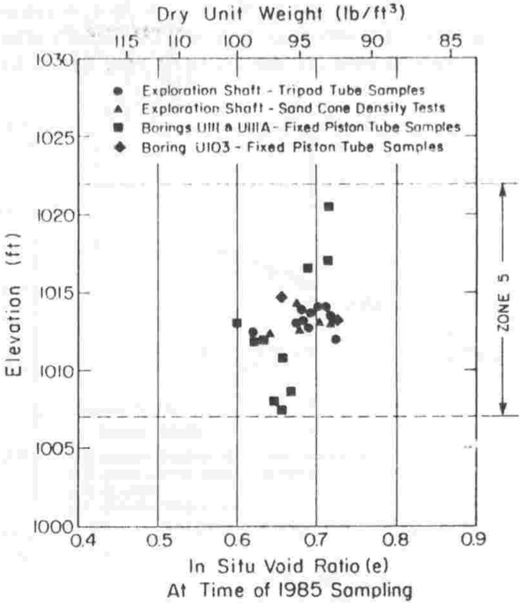

FIG. 8. In Situ Void Ratios within Zone 5 of Hydraulic Fill at Time of the 1985 Field Studies

of the dam made by the Los Angeles Department of Water and Power both prior to and following the 1971 earthquake. These settlements and deflections include: (1) Movements due to the 1971 earthquake; and (2) ongoing consolidation settlements from 1971 through 1985.

A number of assumptions are necessary in order to evaluate how much of the observed surface settlements are due to void-ratio changes that occurred specifically within zone 5 at the base of the downstream hydraulic fill. Two sets of assumptions were selected that led to two methods of analysis that the authors feel are likely to bracket the actual range of possible void-ratio changes that may have occurred within zone 5 during and after the 1971 earthquake. These two analyses, which will be referred to as method A and method B, are somewhat complex and are described in detail by Castro and Keller (1988) and Seed et al. (1988).

The results of these analyses indicate the following postearthquake changes in void ratio in the zone 5 soils of the downstream shell:

- Method A—at location 111, $\Delta e = -0.032$; at location 103, $\Delta e = -0.042$.
- Method B—at location 111, $\Delta e = -0.020$; at location 103, $\Delta e = -0.026$.

Up to this point, all estimates of void-ratio changes have reflected those that occurred in the critical layer on the downstream side of the embankment between 1971 and 1985. It is reasonable to expect that void ratios at the base of the upstream hydraulic fill in 1971 may have been slightly greater than those on the downstream side because of two factors:

1. The upstream soils had been under a lower sustained effective stress due to prolonged submergence prior to 1971.
2. The downstream soils had been subjected to slightly higher effective stresses due to the presence of the 1930 and 1940 berms.

An analysis of the difference in void ratio due to these factors, based on actual soil compressibility data from undisturbed samples, results in a void-ratio difference of up to about $\Delta e = -0.011$ between the zone 5 soils in the upstream and downstream shells (Castro and Keller 1988). This additional void-ratio correction was then added to the void-ratio corrections described previously for method A and method B, to determine the probable void ratios of the soils comprising zone 5 at the base of the upstream shell of the embankment at the time of the earthquake of February 11, 1971.

Determination of Slope of Steady-State Line

The steady-state line (SSL) depicts a correlation, unique for a particular soil, between the void ratio and the effective minor principal stress ($\bar{\sigma}_{3s}$) during steady-state deformation. The effective minor principal stress could be replaced by effective stress on the failure plane ($\bar{\sigma}_{fs}$) or by undrained steady-state strength (S_{us}). Values of $\bar{\sigma}_{3s}$, $\bar{\sigma}_{fs}$, and S_{us} are all related by essentially constant factors that are a function of the effective steady-state friction angle, ϕ_s'.

Evaluation of the slope of the steady-state line represents an important part of the methodology used to evaluate in situ steady-state strengths, because the results of laboratory tests on undisturbed samples are corrected for void-ratio changes associated with sampling and test setup based on assumed parallelism with the SSL established by testing remolded bulk samples. These void-ratio corrections are very sensitive to the slope of the SSL, and small changes in this slope can result in significant changes in the corrected in situ strengths evaluated based on testing of the "undisturbed" samples.

A representative bulk sample of soils from the critical hydraulic-fill layer (zone 5) was made from eight bag samples obtained from the exploration shaft. This bulk sample is referred to as batch mix 7 and is a sandy silt of low plasticity (*ML/SM*). A grain-size distribution curve is shown with a dashed line in Fig. 7 and compared with grain-size curves of undisturbed samples of hydraulic fill from the critical layer. As shown in this figure, the gradation of batch mix 7 falls near the middle of the range of gradations observed in undisturbed samples obtained from zone 5.

Monotonically loaded triaxial shear tests were performed on remolded samples of batch mix 7 at various void ratios to define the slope of the steady-state line for this soil. A variety of testing procedures were used to measure the steady-state strength of this material. The parameters varied in this test program were: (1) Test type (drained and undrained); (2) consolidation type (isotropic and anisotropic); (3) sample preparation procedure (compacted moist, pluviated, or consolidated from slurry); and (4) end-platen treatment (lubricated and rough). Samples were generally placed very loose (at high void ratios) so that they would be contractive during shear.

The steady-state line (SSL) for batch mix 7 is shown in Fig. 9 as a plot of void ratio versus effective stress on the specimen failure plane during steady-state deformation. The results presented in Fig. 9 are those obtained from tests performed at the GEI laboratory. The initial state for each test is also shown in Fig. 9, as well as the path followed during shear.

Three other laboratories also performed triaxial tests on the reconstituted samples of batch mix 7 to define its steady-state line. These laboratories

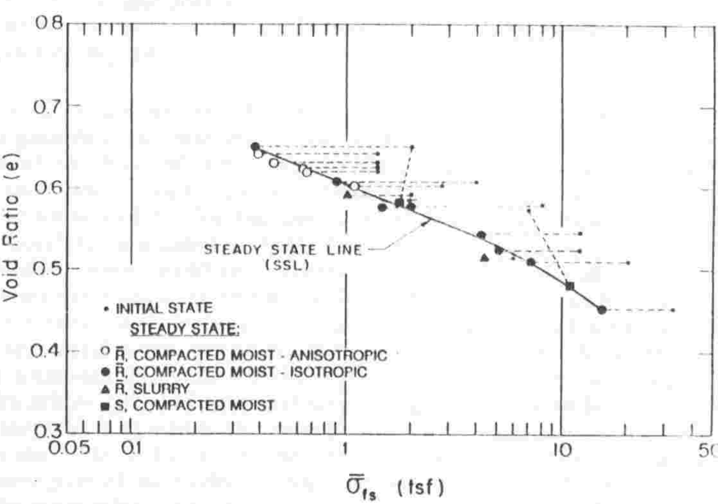

FIG. 9. Steady-State Line for Reconstituted Samples of Hydraulic Fill from Zone 5 of Lower San Fernando Dam

were Stanford University (Seed et al. 1987; Jong and Seed 1988), the U.S. Army Corps of Engineers Waterways Experiment Station, and Rensselaer Polytechnic Institute (Vasquez-Herrera and Dobry 1988). Steady-state strength data from these laboratories are plotted together with the GEI data in Fig. 10. Data from all laboratories plot very close to a unique steady-state line. In general, the agreement between results obtained from the different laboratories is extremely good.

The SSL for batch mix 7 has a straight line portion up to $\sigma_{fs} \approx 4$ tsf, and is slightly curved at higher effective stresses. The straight line portion has a slope of 0.11 on a semilog plot, i.e., the strength changes by a factor of 10 when the void ratio changes by 0.11. The steady-state friction angle for batch mix 7 is approximately 34°.

The various testing methods used to develop the SSL for batch mix 7 show that the slope of the SSL for this material was not affected by the following factors:

- Method of sample preparation, i.e., initial structure or fabric.
- Initial state, i.e., consolidation stress conditions.
- Stress path, i.e., test type.

The slope of the SSL defined for the zone 5 soil was used to correct the strength of "undisturbed" specimens, as described in the following section.

Determination of S_{us} for Undisturbed Specimens

A total of 40 consolidated undrained (R) triaxial tests were performed by GEI Consultants and Stanford University on "undisturbed" samples of the hydraulic fill to define their steady-state strengths. The samples were obtained mainly from location 111 on the downstream side of the dam (see Fig. 5). Two types of samples were obtained at location 111: (1) Fixed-piston samples from borings; and (2) hand-carved tripod tube samples from the exploration shaft. Fixed-piston samples were obtained from borings at all other locations.

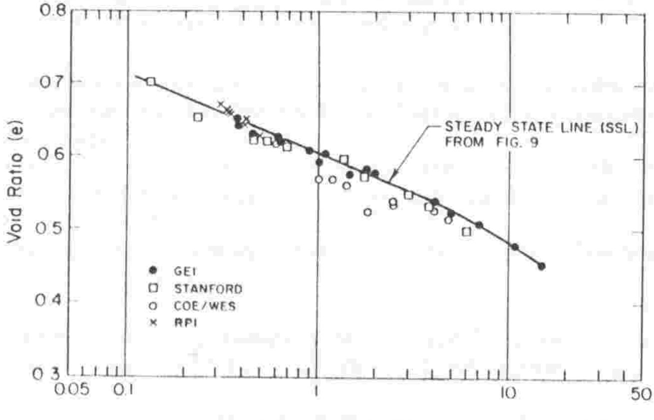

FIG. 10. Steady-State Line for Reconstituted Samples of Hydraulic Fill from Zone 5 Based on Data From Four Laboratories

The undisturbed specimens retrieved were stratified to various degrees. Grain-size analyses of each of the individual undisturbed samples subjected to triaxial testing were therefore performed on a mixture of the layers representative of the failed zone of the actual specimen tested. X-ray photographs of the undisturbed tube samples were examined to select sections of the individual tube samples for triaxial testing that contained primarily only one soil type. Lubricated end platens were used for virtually all tests to allow for the use of shorter samples in order to facilitate the selection of a relatively uniform triaxial specimen with minimal stratification. All samples tested were 2.8 in (7.1 cm) in diameter, with height:diameter ratios in the range of 1.8:1 to 2.5:1.

The specimens were consolidated to relatively high effective stresses so that each specimen would be contractive after consolidation because the steady-state condition is more easily achieved within the strain limits of a triaxial test when specimens are contractive. In most tests, the steady state was approximately reached during the later stages of the test at axial strains of about 10%–20%.

The undrained steady-state strength S_{us} measured in the laboratory corresponds to a "laboratory-tested" void ratio that was significantly lower than the void ratio estimated to have existed in situ in the upstream shell of the dam. Thus, it was necessary to correct the laboratory strengths using the procedures described previously and illustrated schematically in Fig. 2. In making these corrections, the following causes of changes in void ratio were considered:

1. Sample extrusion, setup, and consolidation in the laboratory, with, by far, the largest portion of the change being caused by consolidation.
2. Sampling, which generally resulted in small void-ratio changes.
3. Effects of the 1971 earthquake on the downstream shell and subsequent lowering of the water level within the dam and reservoir. As discussed previously, two methods (method A and method B) were used to estimate the range of void-ratio changes likely to have resulted from these effects.
4. Differences in the stress conditions between the upstream and downstream shells, as previously described.

Fig. 11 illustrates this process of correcting the laboratory-measured values of S_{us} to generate an estimate of the preearthquake in situ S_{us} of the hydraulic fill at the base of the upstream shell for a typical undisturbed sample (in this case sample number GEI-1). The resulting estimated preearthquake values of S_{us} for the upstream hydraulic fill, based on this correction procedure, are listed in Table 1 for all tests that were performed on undisturbed samples. Void-ratio corrections were performed for post-earthquake void-ratio changes estimated by method A and method B, as described previously, in order to encompass the likely range of actual field values. The test results are sorted by the hydraulic-fill zone designations identified at locations 111 and 103 in Fig. 5. The four tests on samples obtained at locations 102, 104, and 105 were assigned zone numbers on the basis of the elevations at which the samples were obtained.

The upstream slide induced by the 1971 earthquake occurred mainly within the lower part of the hydraulic fill, i.e., within the zone 5 soils. Thus, the test results most relevant to the failure are those for the zone 5 samples. The 23 undisturbed samples from zone 5 that were tested ranged from silty sands to sandy silts, with percent fines ranging from 15% to 91% (see Table

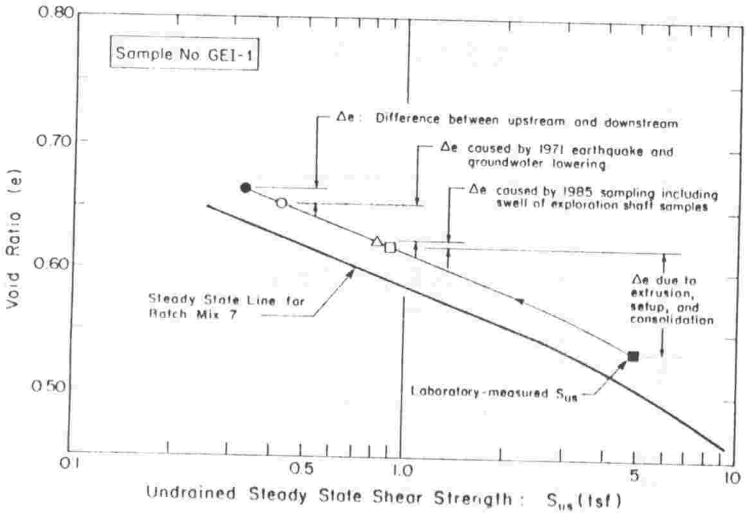

FIG. 11. Correction of Laboratory-Measured Steady-State Strength Data for Effects of Void-Ratio Changes

1.) One of these samples, sample number GEI-12, exhibited considerably higher steady-state strength than the other zone 5 samples, and it was therefore discounted in the analyses of the overall strength of zone 5 soils that follow.

Considering the remaining 22 tests on zone 5 samples, there is still considerable scatter in the data because of the sensitivity of S_{us} to void ratio, and the variations in void ratio and material characteristics inherent in the hydraulic-fill method of deposition. This can be seen in Fig. 12, which presents a plot of estimated preearthquake S_{us} values at the base of the upstream hydraulic fill based on postearthquake void-ratio changes estimated by methods A and B.

The following important conclusions can be drawn from an examination of the results presented in Table 1 and Fig. 12:

1. There is no apparent systematic difference in the data obtained by the two laboratories that tested undisturbed zone 5 samples (GEI consultants and Stanford University).
2. There is no apparent systematic difference between the test results on samples obtained from boreholes and the hand-carved samples obtained from the exploratory shafts.
3. There is no apparent systematic variation of S_{us} as a function of sampling elevation within zone 5.

Fig. 13 is a plot of S_{us} values versus percent fines for the zone 5 soils. As shown in this figure, there is no apparent correlation of S_{us} with percent fines.

The selection of a representative preearthquake steady-state strength value for the upstream zone 5 soils from the data in Table 1 involves a considerable amount of judgment. If one assumes perfect randomness, and in recognition of the fact that failure in the upstream shell seems to have occurred through

TABLE 1. Estimates of Preearthquake in Situ Undrained Steady-State Strengths at Base of Upstream Hydraulic Fill as Computed from Results of Tests on Undisturbed Samples

Test number[a] (1)	Sample location[b] (2)	Sample elevation (ft) (3)	Hydraulic-fill zone (4)	Fines (%) (5)	Estimated Preearthquake In Situ S_{us} Values (lb/ft²) Method A[c] (6)	Method B[d] (7)
GEI-18	B-111	1,007.4	5	91	540	700
GEI-17	B-111	1,008.0	5	45	820	1,060
GEI-16	ES-111	1,008.6	5	35	340	440
GEI-5	B-111	1,010.8	5	40	820	1,060
GEI-20	B-111	1,011.8	5	23	660	840
SU-52	ES-111	1,012.0	5	61	520	680
SU-16	B-111	1,012.0	5	15	160	200
GEI-12[e]	ES-111	1,012.4	5	43	(3,400)	(4,300)
SU-51	ES-111	1,013.0	5	44	600	760
SU-50	ES-111	1,013.0	5	51	840	1,080
SU-4	B-111	1,013.0	5	22	1,140	1,480
GEI-8	B-103	1,013.0	5	44	240	440
GEI-7	ES-111	1,013.1	5	46	900	1,160
GEI-13	ES-111	1,013.4	5	60	400	520
GEI-C1	ES-111	1,013.7	5	53	500	660
GEI-C2	ES-111	1,014.0	5	49	700	900
GEI-1	ES-111	1,014.0	5	44	680	880
GEI-6	ES-111	1,014.1	5	35	320	420
GEI-19	B-103	1,014.7	5	22	300	1,300

GEI-15	B-111	1,016.6	5	62	760	1,000
SU-7	B-111	1,017.0	5	70	660	860
SU-28	B-105	1,019.0	5	43	160	240
GEI-14	B-111	1,020.6	5	90	860	1,120
GEI-C4	B-111	1,030.7	3	92	500	560
GEI-10	ES-111	1,032.5	3	28	1,140	1,280
GEI-2	ES-111	1,032.5	3	—	7,840	8,860
GEI-C5	B-111	1,036.7	2	11	2,300	2,600
SU-11	B-104	1,039.0	3	78	1,000	1,140
SU-10	B-104	1,040.0	3	85	420	480
SU-12	B-111	1,041.0	2	78	120	140
GEI-3	ES-111	1,041.1	2	28	4,460	5,040
SU-45	ES-111	1,042.0	2	84	300	340
SU-46	ES-111	1,042.0	2	4	3,340	3.760
GEI-C3	ES-111	1,042.4	2	8	3,380	3,820
SU-43	ES-111	1,044.0	2	21	460	520
SU-44	ES-111	1,044.0	2	16	1,760	2,000
GEI-4	ES-111	1,044.3	2	96	520	600
GEI-11	ES-111	1,044.3	2	18	3,380	3,820
SU-14	B-102	1,054.0	1	84	620	700

[a]Tests designated GEI performed by GEI Consultants Inc.; tests designated SU performed at Stanford University.
[b]B designates borehole samples taken with a fixed-piston sampler. ES designates hand-carved samples from the exploratory shaft.
[c]S_{us} corrected for postearthquake void-ratio changes estimated by method A.
[d]S_{us} corrected for postearthquake void-ratio changes estimated by method B.
[e]Test number GEI 12 is not included in overall statistical analyses of S_{us} for hydraulic-fill soils in zone 5.

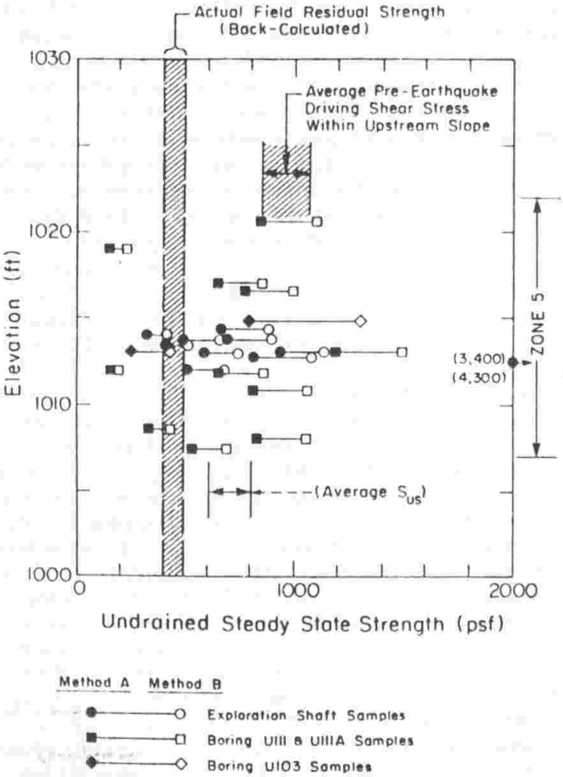

FIG. 12. Estimates of Preearthquake In Situ Steady-State Strengths of Upstream Zone 5 Hydraulic Fill after Correction of Undisturbed Sample Test Data for Effects of Void-Ratio Changes

a zone of substantial thickness, it would be appropriate to consider the average as representative of the strength that can be simultaneously mobilized along the failure zone. On the other hand, the method of deposition of hydraulic fills tends to create a mass of soil with significant layering and anisotropy, and, thus, the resistance that can be mobilized along a failure zone may be somewhat lower than the average. Accordingly, Table 2 presents a compilation of: (1) The average (mean); (2) the mean minus one-half standard deviation; and (3) the mean minus one standard deviation values of the estimated preearthquake S_{us} for the upstream zone 5 soils, based on the range of possible void-ratio corrections as estimated by methods A and B.

The strength values listed in Table 2 should be compared with the best estimate of the actual field strength of $S_{us} \approx 400$ psf to 500 psf as backfigured from the actual slide. The average strength estimates in Table 2 ($S_{us} \approx 610$ psf to 810 psf) are clearly unconservative as compared with the estimated field value. For the strengths developed based on void-ratio corrections by method A, the mean-minus–one–half-standard-deviation value ($S_{us} \approx 490$) is in better agreement with the estimated field strength. For the strengths developed based on void-ratio corrections by method B, the mean-minus-

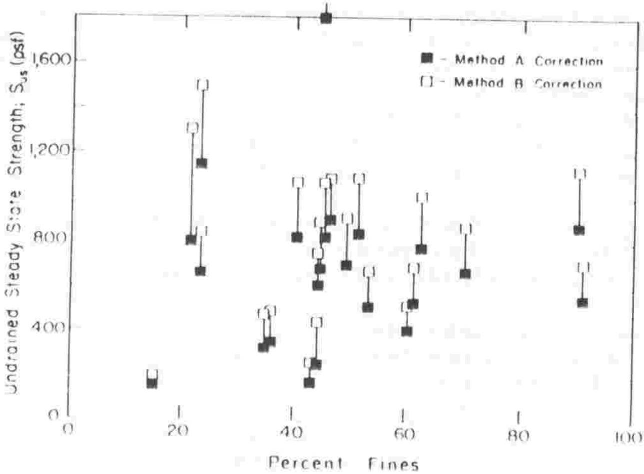

FIG. 13. Estimates of Preearthquake In Situ Steady-State Strengths of Upstream Zone 5 Hydraulic-Fill Soils as Function of Fines Content

TABLE 2. Overall Estimates of Preearthquake S_{us} in Zone 5 at Base of Upstream Hydraulic Fill (Based on 22 Tests of Undisturbed Samples with Corrections for Void-Ratio Changes)

Strength-evaluation level (1)	Basis for Estimation of Postearthquake Void-Ratio Changes for S_{us} Correction	
	Method A (2)	Method B (3)
Average (mean value)	S_{us} = 610 psf (0.305 kg/cm²)	S_{us} = 810 psf (0.405 kg/cm²)
Mean minus one-half standard deviation	S_{us} = 490 psf (0.245 kg/cm²)	S_{us} = 650 psf (0.325 kg/cm²)
Mean minus one standard deviation	S_{us} = 360 psf (0.180 kg/cm²)	S_{us} = 480 psf (0.240 kg/cm²)

one-full-standard-deviation value (S_{us} = 480 psf) is in better agreement with the estimated field strength. The best agreement with the estimated field strength is thus provided by the mean-minus-one–half-standard-deviation to the mean-minus-one-full-standard-deviation values.

The results of this study thus indicate that the methodology employed can successfully predict the potential for the initiation of a liquefaction slide, but that a conservative to very conservative selection of strength (S_{us}) based on the corrected laboratory data is necessary for predicting the full extent of the observed slide movements. In actual practice, the degree of conservatism that is required in the selection of an S_{us} value depends on many factors, e.g., degree of knowledge of soil stratigraphy, including delineation of the zones of weak soils; whether the test data represents average soil conditions or only the loosest soils; the number of tests performed; the estimated reliability of the data, particularly of the in situ void ratio; the consequences of a failure; etc.

It should be noted that regardless of the method used for correcting for earthquake effects (method A or method B), and regardless of whether one selects the mean or the mean-minus-one-standard-deviation strength value, the strength value determined is lower than the preearthquake driving shear stress in the upstream shell, which was estimated to range from about 850 psf to 1050 psf. Thus, however the data are interpreted, it would have been predicted that the upstream section of the dam would have been susceptible to sliding as a result of an earthquake large enough to trigger the failure. However, depending on the assumptions made, the data presented in Table 1 may or may not have predicted how far the slide mass would have moved.

The strengths measured in the upper zones of the hydraulic fill (zones 1, 2, and 3, as shown in Fig. 6) are, on the average, substantially higher than the strength of zone 5 (see Table 1), which is consistent with the fact that the upper part of the hydraulic fill broke into large blocks and moved into the reservoir by riding or "floating" on the liquefied lower part of the hydraulic fill.

There are some limitations of the data that should be noted. The samples of zone 5 were mostly obtained from under the downstream berm, which is the mirror image of approximately the midpoint of the liquefied zone in the upstream 1971 failure. No samples were obtained closer to the slope face where the soil would be expected on average to be somewhat coarser than at the location of the sampling. Only a few samples were tested close to the core where the soil would be expected to be finer. These limitations are believed not to be very significant, since the zone tested is at about the midpoint between the point of hydraulic-fill discharge and the center of the pool (core), and, thus, it is reasonable to assume that the data obtained represents average conditions for the lower part of the hydraulic fill. Furthermore, the soils tested had a wide range of gradations, and gradation differences apparently had no significant systematic effect on the S_{us} values determined in this case (as shown in Fig. 13). Rather, it appears that the S_{us} values were the result of the depositional environment present when the lower part of the hydraulic fill was being placed. Finally, the wide range in values of the test data indicate the need for a significant number of tests to be performed to determine representative values.

The need to correct the laboratory S_{us} values obtained from tests on "undisturbed" samples for void-ratio changes caused by (1) The 1971 earthquake; and (2) the subsequent lowering of the water level in the dam, and also the need to estimate upstream strengths from tests on downstream soil samples, introduced uncertainties that are not normally present in the application of the procedures proposed by Poulos et al. (1985) for determining undrained steady-state strengths of existing soil masses. These additional corrections, which were applied to the Lower San Fernando Dam study, were based on a range of procedures used to estimate the postearthquake void-ratio changes that the authors feel encompass the full range of likely values.

The data presented herein relates to the use of steady-state analysis techniques to address the question of whether or not a slide resulting from soil liquefaction is possible. The determination of the level of earthquake shaking that would have been required to trigger (actually initiate) the failure is not addressed in this paper. The reader is referred to Seed et al. (1988) and Castro and Keller (1988) for the application of methodologies for analysis of triggering of the Lower San Fernando Dam slide movements.

Conclusions

The results of undrained steady-state strength (S_{us}) determinations in the hydraulic fill shells of the Lower San Fernando Dam are in good general agreement with the fact that the upstream shell of the dam was susceptible to the onset of slide movements following the 1971 San Fernando earthquake, but a conservative to very conservative interpretation of the corrected laboratory test data (e.g., the use of a mean-minus-one-half- to mean-minus-one-full-standard-deviation strength value) was necessary in this case in order to provide a reasonable level of agreement with the actual field strength estimated from the observations of the extent and timing of the slide.

Testing techniques to obtain S_{us} values for sands and silts require special care; however, the results obtained independently by four different laboratories in testing of the Lower San Fernando Dam samples were in remarkable agreement.

The undrained steady-state strength varies substantially within a soil deposit because of its great sensitivity to void ratio and, thus, to method of deposition and variations in material characteristics. Thus, the selection of a strength value for analysis requires the exercise of careful engineering judgment based on the performance of sufficient tests and a suitably conservative interpretation of the resulting data.

ACKNOWLEDGMENTS

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