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This Journal is published monthly by the American Society of Civil Engineers. Publications office is at 345 East 47th Street, New York, N.Y. 10017. Address all ASCE correspondence to the Editorial and General Offices at 345 East 47th Street, New York, N.Y. 10017. Allow six weeks for change of address to become effective. Subscription price to members is $10.00. Nonmember subscriptions available; prices obtainable on request. Second-class postage paid at New York, N.Y. and at additional mailing offices. HY, GT.

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11488 PRESSUREMETER IN STRAIN-SOFTENING SOIL

KEY WORDS: Elasticity; Finite elements; Geotechnical engineering; Plasticity; Pressure measurement; Soil mechanics; Strain hardening; Stresses; Strength; Stresses

ABSTRACT: Strain-softening behavior is defined as a gradual loss of shear resistance after a peak strength has been reached. The basic concepts of the theories of elasticity and plasticity are reviewed, and the classical formulation is extended to include analysis of strain softening. A general effective stress-strain-strength model is described, but attention herein is focused on undrained behavior. Closed-form mathematical solutions for stresses, strains, and pore-water pressures around a pressuremeter are worked out, and detailed comparisons are made between the results from strain-hardening and strain-softening behavior. The proposed theory makes it possible to back-figure basic soil stress-strain and strength properties from strain-controlled pressuremeter tests.


11495 ELECTRO-Osmosis AND UNSTABLE EMBANKMENT

KEY WORDS: Clays; Drydocks; Electrochemistry; Electroosmosis; Embankments; Geotechnical engineering; Soil stabilization; Stabilization

ABSTRACT: In the course of construction of a large dry dock project in Singapore, an 8-m high cofferdam embankment was constructed by end tipping decomposed granite material into the sea. At the time of dewatering the cofferdam, large movements of the embankment occurred, and electro-osmosis was chosen as a rapid stabilization technique. The electrode layout, which was chosen after a field test, enabled stabilization to be achieved with a low power consumption of 0.5 kWh/m^2 of soil. The success of this operation has been attributed to the relatively high cationic concentration in the boundary layers of South East Asian clays.


11493 CONSOLIDATION OF FIBROUS PEAT

KEY WORDS: Compaction tests; Geotechnical engineering; Peat; Permeability; Pore pressure; Primary consolidation; Secondary compression; Soil mechanics; Testing

ABSTRACT: A series of single increment vertical consolidation tests have been conducted on undisturbed samples of fibrous peat in 10-in. (250-mm) diam Rowe cell. The results have been assessed in the light of a recently developed theory by Berry and Paskitt. Close agreement is obtained between the observed and predicted rates of settlement and pore pressure dissipation. The decrease in vertical permeability during a consolidation process is of the order of 10^7. As a result, the coefficient of vertical consolidation decreases markedly during a consolidation increment.

$K_i$ = stiffness of system with embedment to top of $i$th layer;
$K_x, K_z, \ldots$ = stiffness coefficients for displacement in $x, z, \ldots$ directions;
$M$ = number of layers that define embedment;
$N$ = total number of layers;
$P$ = applied force;
$r_e$ = radius of circular footing;
$S_i$ = stiffness of $i$th layer;
$S_j$ = stiffness of $j$th embedded layer;
$\delta$ = displacement; and
$\nu$ = Poisson's ratio.

Subscripts
$i, j$ = layer indices; and
$x, z, \psi, \theta$ = directions of displacements.

**Journal of the Geotechnical Engineering Division**

**Properties of Soil in the San Fernando Hydraulic Fill Dams**

By Kenneth L. Lee,¹ H. Bolton Seed,² Izzat M. Idriss,³ Members, ASCE, and Faiz I. Makdisi,⁴ A. M. ASCE

**Introduction**

The unfortunate slides that damaged the Upper and Lower San Fernando hydraulic fill dams during the February 9, 1971 San Fernando earthquake provided the incentive for a thorough investigation to be made. A companion paper (19) describes the construction of these two dams and the observed behavior during the earthquake. This paper describes the properties of the soils as determined from field drilling, sampling, and laboratory tests. Another companion paper (20) describes the seismic stability analyses that were made of these two dams. Additional details regarding the study are available elsewhere (18). The two San Fernando dams are located in the upper part of the San Fernando Valley of greater Los Angeles, Calif. The Lower Dam was constructed first, beginning in 1912, followed by the Upper Dam beginning in 1921.

**Description of San Fernando Dams**

The dams are located within about 2 miles of each other. They are each founded on similar natural alluvial soils and were both constructed primarily by hydraulic filling using similar borrow material.

Fill for the Lower Dam was loosened in the borrow area by hydraulic jets of water and transported to the site by means of wooden sluice troughs. Dikes of dry fill were placed at the outer edges of the dam to contain the soil and

Note.—Discussion open until January 1, 1976. To extend the closing date one month, a written request must be filed with the Editor of Technical Publications, ASCE. This paper is part of the copyrighted Journal of the Geotechnical Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 101, No. GT8, August, 1975. Manuscript was submitted for review for possible publication on May 3, 1974.

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water slurry that was discharged from the sluice troughs at both the upstream and downstream edges. After the dam had been constructed to about two-thirds of its final height, rolled fill was added with minimal compaction to bring the dam to the desired height. Finally, many years later, a downstream berm of rolled fill was added.

The Upper San Fernando Dam was constructed by a semihydraulic fill method. The fill was excavated dry, hauled to the site by wagons, and dumped over the side of the dike into the ponds. From there it was sluiced into place by hydraulic jets operating from barges floating within the central pool of the dam. Later, a small addition of rolled fill was added at the top. Because the rolled fill portions of the dams had very little influence on the overall stability, the major part of the investigation was directed to determining the properties of the hydraulic fills.

Typical cross sections of the two dams are presented elsewhere (19). The Lower Dam was constructed on about 35 ft (11 m) of alluvium and rose to a maximum height of about 140 ft (43 m) above the original ground. The Upper Dam was built on about 40 ft–60 ft (12 m–18 m) of alluvium and rises to a maximum height of 80 ft (24 m) above the valley floor. A total of 35 drill holes were made through the fill and alluvium and into the bedrock at the Upper and Lower Dams. These were made with a Failing 1500 rotary drill using a 4-1/2-in. (110-mm) diam tri-cone bit and bentonite drilling mud in uncased holes. Three inch diameter samples were obtained at 5-ft (1.5-m) intervals. In the hydraulic fill these were obtained by pushing 3-ft (0.9-m) long thin-wall Shelby tubes ahead of the hole. In the harder zones of alluvium and some of the sedimentary rock, a Pitcher sample was used with the Shelby tubes.

Standard penetration tests were made at 5-ft (1.5-m) intervals in each of the borings between each undisturbed sample. The hydraulic fill soil graded from a coarse silty sand at the edges to a clay core in the center. In the hydraulic fill of the Lower Dam, the blow counts range from about 10 blows/ft–25 blows/ft (33 blows/m–83 blows/m), whereas at the Upper Dam the blow counts in the hydraulic fill were a little less. The alluvium foundations consisted of a silty sandy gravel mixture with occasional lenses or pockets of more clayey material. In the alluvium foundations at both dams, the blow counts were somewhat greater than in the hydraulic fill, but were very erratic.

After studying the nature of the slide debris of the Lower Dam and viewing many of the Shelby tube samples, it appeared that much of the soil in both the upstream and central portion of that dam had been disturbed by the slide and was not representative of the soil prior to the earthquake. Therefore, Shelby tube samples from the upstream and central portion of the Lower Dam were given only visual classification and not subjected to detailed laboratory testing. However, Shelby tube samples from the downstream portion of the Lower Dam and from all of the Upper Dam were considered to be relatively undisturbed by the earthquake and were tested in the laboratory for density and strength properties.

In addition to the drill holes previously mentioned, three 24-in. (610-mm) diam bucket auger holes were drilled to a depth of about 35 ft (11 m) into the hydraulic fill from the downstream berm of the Lower Dam. Field densities and undisturbed samples were obtained from these holes as well. Finally, a very large trench excavation was made through the central and upstream portions of this dam to give access to, and a visual impression of, the interior of the dam. Some field density tests were made in the walls of this trench in zones of soil that appeared not to have been disturbed during the slide.

In addition to the bore holes, four test trenches were also excavated along the full width of the downstream berm of the Upper Dam; these trenches were extended to the downstream toe. Although they were only 6 ft–8 ft (1.8 m–2.4 m) deep, the trenches extended well into the undisturbed hydraulic fill, providing direct access to it, and a good visual impression of the nature of the hydraulic fill material.

![FIG. 1.—Soil Stratification along Wall of Inspection Trench, Upper San Fernando Dam](image)

![FIG. 2.—Distribution of Grain Sizes in Hydraulic Fill from Outer Shell towards Center of Embankment, Upper San Fernando Dam (1 ft = 0.3048 m)](image)

A photograph of the wall of one of the trenches in the Upper Dam is shown in Fig. 1. The end of the trench, where a man is standing, is near the center of the dam and the trench extends back toward the downstream shoulder of the berm. The soil stratification shown in this photograph is typical of that observed in each of the trenches in the Upper Dam. In the outer part of the
dam away from the core, the hydraulic fill consists of alternate layers of clean sand, silty sand, and sandy silt. The thickness of the layers range from a few inches up to about 1 ft (0.3 m).

The general gradation of the soil changes from coarse to finer materials with increasing proximity to the center of the dam. This change in average gradation with location along the trench is indicated by the grain-size curves from five samples taken at various locations along the test trench and shown in Fig. 2. Unfortunately, because the dam had to remain in service, it was not possible to extend the trench all the way to the upstream face. However, undisturbed samples taken from the center and the upstream portion of the dam, indicate that the soil became even more clayey in the center of the dam, as expected for hydraulic fill embankments.

One 60-ft (18-m) deep trench was also excavated through part of the Lower Dam. The nature of the soil exposed on the walls of this trench is examined elsewhere (18,19). In the undisturbed parts, the soil was highly stratified, similar to that shown in Fig. 1 for the Upper Dam.

**GRAIN-SIZE DISTRIBUTION**

Grain-size determinations were made for many of the samples tested at both the Upper and Lower Dams. A summary of these grain-size distribution curves for the samples of hydraulic sand fill is shown in Fig. 3. As the sample tubes were opened, each of the samples were visually classified as being either coarse sand, fine sand, or clay material. Because previous studies had indicated that clayey soils are stronger than silts or sands under cyclic loading conditions, the laboratory testing was largely confined to the coarse and fine sandy materials. The foundation alluvium was a little coarser and a little better graded than the hydraulic fill material.

**FIELD AND RELATIVE DENSITIES**

For the purpose of correlation and for comparing the data with that obtained at other sites, it was important to establish the relative and the absolute densities of the soils encountered at these dams. A number of different types of tests were used in this regard.

Field densities were determined by the sand cone method at several locations in the test trenches and in the bucket auger holes. In addition, sufficient soil at each field density location was taken to the laboratory for maximum and minimum density determinations. The minimum density tests on these samples were performed according to the American Society for Testing and Materials (ASTM) designation D2049-69. This method consists of pouring oven-dry soil through a standard funnel into a steel mold. Maximum density tests were performed on these samples using the modified American Association of State Highway Officials (AASHO) (1961) Procedure. This is the well-known impact compaction method; 1/30-cu ft (0.001-m³) mold; five layers; 25 blows/layer with a 10-lb (4.5-kg) hammer; and a 18-in. (460-mm) drop.

The results of these minimum, maximum, and field density tests on the hydraulic fill soil from the Lower Dam are shown in Fig. 4. Although the data are scattered, there is a definite trend of decreasing density with decreasing mean grain size.

Field densities were also determined by direct measurement from the undisturbed 3-in. (76-mm) diam Shelby tube samples. These data for the Lower Dam are shown, together with the sand cone data, in Fig. 5. Data for the Upper Dam were similar (18). The data from either method of density determination are essentially the same in terms of scatter or actual values. The agreement suggests that the individual data from each source may be correct and the scatter represents the natural variation from one point to another. Because of this variation from sample to sample and from point to point in the field, it was felt that relative densities would be better evaluated on an average basis.
than by direct comparison for individual samples. Therefore, visual best fit lines were drawn through the data points of field density for the Upper and Lower Dams as shown in Fig. 5 to represent the average field density corresponding to the particular mean grain size. 

By definition, the maximum density of a soil is the maximum density to which it can be compacted without significantly crushing or breaking the individual grains. The Modified AASHO compaction tests performed in connection with the sand cone field density tests did not necessarily give the maximum density of the sample. Therefore, vibratory compaction tests using both wet and dry soil were also performed on special samples to better define the actual maximum densities. Unfortunately, the quantity of soil obtained from the individual Shelby tube samples was insufficient for performing the impact and vibratory compaction tests. Therefore, as a compromise for the Shelby tube samples, similar soil from a number of tubes was mixed together to form a single composite sample on which these comparative maximum density tests were performed.

The result of these comparative maximum density tests on the composite samples indicated the following:

1. The Modified AASHO density for the composite samples was significantly greater than the Modified AASHO densities for individual samples. It appeared that mixing several samples together improved the compaction characteristics, possibly by improving the grain-size distribution.

2. The maximum vibrated density of composite samples was about 5pcf-6pcf (80 kg/m³-96 kg/m³) greater than the maximum density obtained by the Modified AASHO procedure. For the fine soil, the highest densities were obtained by vibrating the soil in a dry condition, whereas for the coarse soil, the highest densities were obtained by vibrating the soil in a wet condition.

Additional data on the effects from different types of compaction tests on granular soils have been reported by Felt (2).

Because the density tests performed on the composite samples gave values significantly higher than individual field samples, it was not considered appropriate to use the actual maximum densities from composite samples as a basis for determining the relative density of the soil in the field. However, because it was observed that the maximum vibrated densities of the composite samples were approx. 5pcf-6pcf (80 kg/m³-96 kg/m³) greater than the maximum density obtained by the Modified AASHO procedure, it was assumed that approximately the same difference would probably be found for individual field samples. Therefore, the actual maximum densities for the field conditions were selected as 5pcf-6pcf (80 kg/m³-96 kg/m³) greater than the average curve of maximum densities obtained by the Modified AASHO procedure [Figs. 6(a) and 6(b)].

Because some agencies prefer to base compaction specifications on the Standard AASHO density, maximum densities were also obtained using this laboratory impact compaction procedure (1/30 cu ft (0.001-m³) mold; 5.5-lb (2.5-kg) hammer; 12-in. (300-mm) drop; three layers; and 25 blows/layer) [Figs. 6(c) and 6(d)].

The minimum density of a soil is, by definition, the lowest stable density to which a soil may be placed without artificial means such as bulking in a moist condition. Four different procedures were used to evaluate the minimum density values that would be appropriate for these soils:
1. ASTM designation D2049-69, pouring oven-dry through a standard funnel.
2. Dry pouring similar to ASTM, but using a funnel with a smaller opening.
3. Oven-dry slowly stirring and rotation in a 1,000-cc flask (5).
4. Sedimentation. Mix four to five 100-g batches in water and pour separately into a 1,000-cc flask of water. Allow each to settle before adding the next.

![Graphs and data from figures 7(a) to 7(c) and 7(e) to 7(f)]

**FIG. 7.** Summary of Density and Compaction Data (1 lb/ft³ = 16 kg/m³)

**FIG. 8.** Relative Densities Determined from Standard Penetration Test Data, Upper San Fernando Dam (1 ft = 0.3048 m)

The results of these tests are summarized in Figs. 6(e) and 6(f). The lowest densities were obtained from the sedimentation procedure and the highest densities being obtained by the ASTM oven-dry procedure. Because the sedimenting procedure led to a highly segregated soil with layers of very loose silt separating layers of sand, all at essentially zero overburden pressure, it was felt that the minimum densities obtained by this procedure were probably too low. In contrast to the observations from the maximum density tests on composite and on individual samples, there appeared to be no distinction between the minimum density results from composite samples as compared to individual samples.

Data from each of the minimum, maximum, and field density series of tests for the hydraulic fill and the alluvium at both dams showed a similar pattern to that presented in Fig. 5. Although there was scatter, the data for each series could be represented by an average curve that showed a decrease in density with decreasing grain size (18) (Fig.6). A comparison of the best fit lines for the minimum, maximum, and field densities of the hydraulic fill at the Upper and Lower Dams is shown in Figs. 7(a) and 7(b).

From these curves of average densities, the average relative density at any grain size may be calculated from

\[
D_r = \frac{\gamma_d \gamma_{d_{\text{max}}} - \gamma_{d_{\text{min}}}}{\gamma_d \gamma_{d_{\text{max}}} - \gamma_{d_{\text{min}}}} \quad (1)
\]

The variation in calculated average relative density for the hydraulic fill in the Upper and Lower Dams is shown in Figs. 7(c) and 7(d). These values range from about 51%-58% for each dam. They are similar to relative density values for other hydraulic fills reported by Turnbull (25) and by Whitman (26) (45%-60%).

The relative compaction of a soil is defined as the ratio of the field density to the maximum density obtained by some standard laboratory procedure. Values of relative compaction based on both the Standard and the Modified AASHO compaction tests are shown in Figs. 7(e) and 7(f). The relative compaction based on the Standard AASHO test ranges from about 92%-100%, whereas the relative compaction based on the Modified AASHO test range from about 83%-93%. The variation and spread between minimum and maximum density values for different grain sizes in the hydraulic fills is shown in Figs. 7(e) and 7(f). These data are similar to data from other soils summarized by Lee and Singh (12).

Similar procedures to those previously described for the hydraulic fill were used to determine the actual and relative densities and the relative compactions of the alluvial materials forming the foundations of the Upper and Lower Dams. Fewer data were available for the alluvium, but the trends were similar to those established for the hydraulic fills. The densities showed a tendency to decrease with decreasing grain size. In addition, the field densities at both the Upper and Lower Dams were very similar, and ranged from a maximum of about 117 pcf (1,870 kg/m³) for soil with \( D_{90} \) equal to 0.03 mm. The average relative density of the alluvium ranged from about 65%-70%.

As mentioned previously, standard penetration tests were performed at 5-ft (1.5-m) intervals in each of the drill holes made through the hydraulic fill and alluvium at these two dams. Using the Gibb's and Holtz (4) correlations, relative
density values were determined for each Standard Penetration test. These data for the Upper Dam are combined into four different charts in Fig. 8 showing relative density at different depths below the ground surface at four different locations across the dam: upstream slope, crest, downstream berm, and downstream slope. Similar relative density data for the Lower Dam are presented in Fig. 9.

Although there is scatter among the individual data points, taken together the data at any one relative position across either dam shows fairly well-defined trends. The relative densities in the upper rolled fill zones are significantly

<table>
<thead>
<tr>
<th>Soil zones (1)</th>
<th>Upper Dam</th>
<th>Lower Dam</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Depth, in feet</td>
<td>$V_s$, in feet per second</td>
</tr>
<tr>
<td>Hydraulic fill</td>
<td>40</td>
<td>600</td>
</tr>
<tr>
<td>Alluvium—Upper Alluvium—Lower Rock</td>
<td>100</td>
<td>1,500–1,800</td>
</tr>
</tbody>
</table>

1 ft = 0.3048 m; 1 fps = 0.3048 m/s.

FIG. 9.—Relative Densities Determined from Standard Penetration Test Data, Lower San Fernando Dam (1 ft = 0.3048 m)

higher than the relative densities in the hydraulic fills. Some of the data indicated relative densities for the alluvium of approx 100%. This was probably caused by the penetration spoon encountering a large piece of gravel, because the alluvium was very heterogeneous and contained coarse gravel and cobbles as well as pockets of sands, silts, and clays.

At the downstream portion of the Lower Dam which did not participate in the landslide, the relative densities indicated by the standard penetration blow count following the earthquake were about the same as the relative densities indicated by standard penetration tests performed in a drill hole made prior to the earthquake [Figs. 9(b) and 9(c)]. Thus, there appeared to be no significant earthquake-induced densification in this zone of relatively loose granular hydraulic fill. Note that Silver and Seed (23) and Lee and Albaia (7) have presented data that indicated that loose granular soils may compact by about 0.5% during a strong earthquake. This would be equivalent to changing the relative density by only about 10% which would be much too small to be detected by standard penetration or other field density testing procedures.

Comparing the relative density data obtained by the standard penetration tests from the data described previously obtained from field and laboratory testing, indicates that both the range of scatter and the actual average values of relative density in the hydraulic fill and the alluvium are similar for each type of test. Many authors have described the applications of relative density to various engineering problems, and the limitations of the methods used to evaluate the relative density of a soil. The proceedings of a recent comprehensive conference on this subject have been summarized in Ref. 22. Although it is well known that individual determinations of relative density of any method are prone to significant errors, the concept of relative density nevertheless continues to have merit and is useful in expressing general trends in the performance of granular materials. There is no doubt that some of the inherent errors may be reduced by improving testing standards and techniques. However, it would appear that soils in the field, particularly soils placed by hydraulic filling or other sedimentation processes, are likely to possess significant random variations over very short distances. In these cases, if only a few relative density determinations are made even at fairly close proximity to each other, the results may show such a wide scatter as to be misleading or of little value. However, the results of this study would seem to indicate that the trends indicated by a large number of tests, even in variable soils such as hydraulic fills, may be sufficiently well defined to be useful for correlation and comparative purposes.

**Seismic Surveys**

The seismic response analysis described in a companion paper (19) required data for the dynamic shear moduli of the various soils at the two dams. Seismic surveys were made using cross hole measurements between adjacent drill holes to determine the compression and shear wave velocities in the embankment and foundation soils at different depths. The shear modulus, $G$, can be readily computed from the shear wave velocity, $V_s$, from the equation

$$G = \frac{\gamma V_s^2}{g}$$

(2)

in which $g$ = acceleration of gravity; and $\gamma$ = unit weight of the soil.

For analysis purposes, the shear modulus of granular soils may be expressed by the relationship

$$G = 1000 K_2 \sigma_m^{1/2}$$

(3)

in which $K_2$ = a soil constant that depends on the type and density of the soil and the strain amplitude of the shear deformations; and $\sigma_m$ = the mean effective pressure acting on the soil (14). Values of shear modulus computed
from the shear wave velocity measurements correspond to very low shear strains, are the highest values of dynamic shear modulus, and correspond to $K_z = K_{2\text{max}}$ in Eq. 3. Values of $V_s$ and $K_{2\text{max}}$ for the major soil zones in the two dams are shown in Table 1.

**SOIL STRENGTH UNDER STATIC LOADING CONDITIONS**

Even in a seismic stability study some static loading strength properties of the soils are required. Therefore, a large number of static loading triaxial tests were performed on undisturbed 3-in. (76-mm) diam Shelby tube samples of the hydraulic fill sands and the alluvial materials.

All of the tests were performed on isotropically consolidated samples at effective confining pressures ranging from 1 ton/sq ft to 4 tons/sq ft (56 kN/m$^2$–380 kN/m$^2$) and sufficient back pressures were used to insure complete saturation. Both consolidated-drained and consolidated-undrained tests with pore pressure measurements were performed. Failure was defined as the peak axial stress developed during the test.

The results of typical drained and undrained tests on hydraulic fill samples from the Upper and Lower Dams are shown in Figs. 10 and 11. It is recalled that these samples had a relative density of about 55%, yet note that in both cases the drained and the undrained test samples showed a tendency to dilate slightly before and after failure. Although the peak strength is clearly defined, there is no major loss in strength for large strains following the peak stress conditions. These data are typical of the stress-strain relations observed from

![Graph](image)

**TABLE 2.—Summary of Static Shear Strength Parameters**

<table>
<thead>
<tr>
<th>Dam and Soil</th>
<th>Total Stress Undrained Tests</th>
<th>Drained and Undrained Tests</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$c$, in tons per square foot</td>
<td>$c'$, in tons per square foot</td>
</tr>
<tr>
<td>Upper Dam</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hydraulic fill</td>
<td>0.55</td>
<td>0.0</td>
</tr>
<tr>
<td>Alluvium</td>
<td>0.71</td>
<td>0.0</td>
</tr>
<tr>
<td>Lower Dam</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hydraulic fill—average</td>
<td>1.02</td>
<td>0.0</td>
</tr>
<tr>
<td>Hydraulic fill—conservative</td>
<td>0.6</td>
<td>0.0</td>
</tr>
<tr>
<td>Alluvium</td>
<td>0.78</td>
<td>0.0</td>
</tr>
</tbody>
</table>

1 ton/sq ft = 95.8 kN/m$^2$.

![Graph](image)

**FIG. 10.—Typical Stress-Strain Curves in Static Loading Triaxial Tests on Samples of Hydraulic Fill, Upper San Fernando Dam**

over 120 static triaxial tests that were performed on the hydraulic fill and alluvial soils from these two dams.

When the peak strengths were plotted on modified Mohr diagrams, the total stress data were found to show some scatter. However, reasonably well-defined best fit straight lines could be drawn through the data to indicate the friction

![Graph](image)

**FIG. 11.—Typical Stress-Strain Curves in Static Loading Triaxial Tests on Samples of Hydraulic Fill, Lower San Fernando Dam**
angle and the cohesion intercept on a total stress basis. On the other hand, very little scatter was observed in the effective stress data and there was no distinction between the results from drained and undrained tests.

Each of the samples tested was visually classified as being either coarse sand or fine sand. When the data were plotted there was no observable distinct-

### TABLE 3.—Soil Parameters Used in Nonlinear Static Strength Analyses

<table>
<thead>
<tr>
<th>Soil Parameter (1)</th>
<th>Symbol (units) (2)</th>
<th>Upper Dam</th>
<th>Lower Dam</th>
<th>Upper Dam</th>
<th>Lower Dam</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Hydraulic fill</td>
<td>Alluvium</td>
<td>Hydraulic fill</td>
<td>Alluvium</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(3)</td>
<td>(4)</td>
<td>(5)</td>
<td>(6)</td>
</tr>
<tr>
<td>Dry unit weight</td>
<td>$\gamma_d$, in pounds per cubic foot</td>
<td>120</td>
<td>67</td>
<td>106</td>
<td>110</td>
</tr>
<tr>
<td>Buoyant unit weight</td>
<td>$\gamma_b$, in pounds per cubic foot</td>
<td>60</td>
<td>68</td>
<td>64</td>
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1 pcf = 16 kg/m³.

### Figures

**FIG. 12.—Results of Cyclic Load Tests on Isotropically Consolidated Samples of Hydraulic Sand Fill, Upper San Fernando Dam**

**Static Undrained Strengths of Clay Soils**

As described previously, there were numerous thin layers of clay within the hydraulic fill shell and in addition the central core of each dam was predominately clay material. This clay was insensitive, in that it did not loose strength on remolding. Because previous studies have shown that under cyclic loading conditions clays are stronger than saturated sands at corresponding densities (9), it was considered that the clay did not play a significant part in causing the seismic instability of the dam. Therefore a comprehensive testing program was not carried out on the clay soil.

Nevertheless, a fairly large number of small torque shear tests were performed on the larger clay seams that were found in the Shelby tubes and on some clay seen exposed in the inspection trenches. When the measured shear strength, $Su$, was plotted versus the effective overburden pressure, $p$, a fairly consistent relationship was found with $Su/p = 0.24$ for the clay below the water table at the Upper Dam. Above the water table, the shear strengths increased due to the effects of desiccation. A small number of tests performed on clay soils from the Lower Dam suggested that $Su/p$ for this material was probably slightly higher than at the Upper Dam.

Atterberg limit tests were performed on several samples of the clay found in the hydraulic fill at both dams. The data plotted above the A-line. The liquid limit ranged from about 37-60 and the plasticity index values ranged from about 20-40. There was no clear distinction between the clays from the Upper and Lower Dams. According to the empirical relation (24)

$$Su = 0.11 + 0.0037 PI$$

the $Su/p$ ratio for the clay in these dams would be expected to range between about 0.18 and 0.25 for normally consolidated soils. This correlates reasonably well with the measured value of 0.24.

**Strength of Soil under Cyclic Load Conditions**

As described elsewhere (20) seismic stability analyses of earth dams require...
a knowledge of the strength of the soils under cyclic loading conditions. Under the present state-of-the-art this is conventionally done using cyclic loading triaxial tests. Several series of tests are required using groups of samples consolidated under different effective stresses representative of the effective stresses found at different locations within the dam. The laboratory data are then grouped in a convenient manner to facilitate interpolation to determine the cyclic loading strength of elements at the desired locations in the field. It has been found convenient to perform triaxial tests in groups of different values of the effective confining pressure, $\sigma_{3e}$, and in groups of the different anisotropic consolidation stress ratios $K_c = \sigma_{v3}/\sigma_{3e}$ of 1.0, 1.5, and 2.0. After the samples have been completely saturated and consolidated to the desired effective stress conditions, they are subjected to a train of uniform cycles of axial stress increase and decrease symmetrically about the static axial stress condition. This cyclic loading is continued until the sample fails. Then, in preparing the data for use in an actual seismic stability analysis, appropriate correction factors are required to account for the differences between the uniform cyclic loads in the laboratory triaxial tests and the actual erratic shear stress history produced by an earthquake on an element of soil in the field (8,21).

A total of some 90 pulsating loading triaxial tests were performed on undisturbed 3-in. (76-mm) diam Shelby tube samples of the granular soils from the hydraulic fills and alluvium at the Upper and Lower Dams. The tests were performed using a square load shape at 1 Hz, applied by a pneumatic piston.

Following procedures developed in previous studies (13,16), it has been found convenient in analyzing the cyclic loading tests to plot the peak strain amplitude and peak pore pressure data for each test on semilog paper. Data for three typical isotropically consolidated tests are shown in Fig. 12. The accumulative axial strains shown on Fig. 12(a) are the peak to peak, or double amplitude strains that were recorded at each cycle. For reference purposes, the cyclic axial stresses applied to each sample are also given. Using these data, it is a straightforward matter to prepare the cross plot shown on Fig. 12(c) which presents the pulsating deviator or axial stress as a function of number of cycles required to cause a certain specified response in the sample. For example, the solid dots represent the conditions required to cause liquefaction as defined by an excess pore pressure equal to the effective confining pressure. The open circles represent the stress conditions required to cause 5% double amplitude strain, or $\pm 2.1/2%$ single amplitude strain. Similarly, the dash line through the open squares represents the cyclic stress conditions required to cause $\pm 5%$ single amplitude strain.

Data from each of the pulsating loading tests were plotted as shown in Fig. 12. For the anisotropically consolidated samples, $K_c > 1.0$, the axial strains were defined either in terms of single amplitude (i.e., one-half the peak to peak) strain or the maximum accumulative strain, whichever was greater. Thus, to be consistent, in the subsequent analyses the pulsating loading strengths were defined as the cyclic stress required to cause a certain specified value of axial strain regardless of whether or not liquefaction developed during the pulsating loading test. Generally, it was found that liquefaction always developed for isotropically consolidated samples and for anisotropically consolidated samples where the pulsating stress was large enough to cause a stress reversal during each cycle. Liquefaction generally did not develop for anisotropically consolidated samples where no stress reversal occurred. This agrees with trends noted from previous studies (11,16).

The results from one series of tests on the hydraulic sand fill at the Upper Dam are shown in Fig. 13 along with results from tests at the same consolidation pressure conditions for the alluvium at the Upper Dam. Each of these two series of tests contained more data points than usual, because these were the first tests performed in this study. Approximately one-half of the tests for each soil are for the fine sands and the other half are for the coarse sands. When the data were plotted together, there was no apparent distinction between the results from the fine and the coarse sands. However, despite some scatter that is normal for pulsating loading tests on undisturbed samples of soil, the alluvium clearly has a higher pulsating loading strength than the hydraulic fill. This difference in cyclic loading strength is commensurate with the differences in average relative densities of the two types of soil.

A similar comprehensive set of tests were performed on isotropically consolidated samples of hydraulic fill sand and alluvium at the Lower Dam. These data also showed that the alluvium was significantly stronger than hydraulic...
As a result of these findings and because it was desired to test only the weaker soils that would be the first to fail during the earthquake, no further cyclic loading tests were performed on samples of the alluvium from either dam.

As described elsewhere, one of the important steps in a seismic stability analysis is to determine the equivalent number of uniform cycles of stress that are produced by the design earthquake (8). After this has been done, it is a straightforward matter to pick off appropriate strengths from plots as shown in Fig. 13 for the appropriate number of cycles. Strength data from several series of tests can then be combined into a single diagram showing cyclic loading strengths for different consolidation stress conditions. An example of this type of strength diagram for the hydraulic fill sand is shown in Fig. 14. For these data, failure has been defined as the peak stress during cyclic loading that would cause 5% strain in 5 cycles. Data for other possible conditions are presented elsewhere (18). All of these data are consistent with the results from other studies (3,10,11,15,16) showing that the pulsating loading strength increases with increasing consolidation pressure and increasing the $K_s$ ratio. The strength envelopes show some nonlinearity, tending to flatten a little with increasing consolidation pressure. The data show that the seismic loading strength of the hydraulic fill from the Lower Dam was always slightly greater than the cyclic loading strength of the hydraulic fill from the Upper Dam.

From the test data such as shown in Fig. 14 certain additional corrections and manipulations are required before they can be used directly in the stability calculations. These additional manipulations are more appropriately described along with a description of the stability analyses (20).

**Summary and Conclusions**

This paper has summarized the results of field and laboratory investigations of the properties of the hydraulic fill and alluvial foundation soils at the Upper and Lower San Fernando Dams. The purpose of the studies was to provide the data required for the seismic stability analyses that were made following the 1971 San Fernando earthquake, to explain the type of failures that were observed. A description of these failures and of the subsequent seismic stability analyses are presented elsewhere (19,20). The purpose of presenting this soil property data is to provide and disseminate this information with the hope that it may be useful for comparative purposes with data from other soils.

Each of these two dams was constructed by hydraulic filling techniques using soil obtained from the natural alluvium and the soft sedimentary rock near the sites. As indicated by the grain-size distribution curves, the hydraulic filling process caused a considerable amount of sorting of materials with the result that the hydraulic fill at the outer part of the dam consisted of well-defined alternate layers of fine silty sands and coarse clean sands. The sorting became less distinct and the grain sizes became finer towards the center of the dams, and in the central zone there was a wide core of medium to highly plastic clay. In contrast, the natural alluvial foundation soils were not noticeably layered, but were very heterogeneous with the average grain-size distribution curves showing somewhat better gradation than for the hydraulic fills.

The minimum, maximum, and field densities of all soils showed significant scatter from one test to the next, but when all the data were viewed together there was a well-defined relation showing a decrease in any of the densities with decreasing mean grain size. The relative density of the alluvium at each dam site was about 65%-70%. From point to point, the relative density of the hydraulic sand fill varied between extreme limits of about 40%-70% but the average values ranged between about 51%-58%. The degree of compaction based on the Modified AASHO (1961) test procedure ranged from about 85%-92% at each dam. The degree of compaction based on the Standard AASHO test procedure ranged from about 93%-100% for each dam. The average relative densities calculated from the Standard Penetration Test blow count using the Giebe and Holtz (4) procedures, were approximately the same as the relative densities obtained by the classical direct methods.

The results of field seismic surveys, field density tests, laboratory static strength tests, and laboratory cyclic loading tests are all consistent in showing that the alluvial soil in the foundation of the dams is stiffer and stronger than the hydraulic fill materials. The hydraulic fill soils at each dam showed normal stress behavior under static loading conditions. The soils showed a tendency to dilate at failure and gave no indication that liquefaction would be produced under static loading conditions. However, under cyclic loading conditions, large peak excess pore pressures developed with the result that many samples failed by liquefaction. The cyclic loading strengths of these soils are consistent with the cyclic loading strengths obtained from tests on other soils at similar stress and relative density conditions.

**Acknowledgments**

The investigation described in this report was sponsored cooperatively by the State of California Department of Water Resources (Division of Safety of Dams), the Los Angeles Department of Water and Power, and the National Science Foundation.

The Department of Water and Power was responsible for field work at the dam sites including borings and sampling, trench sections, field density determinations, and associated laboratory tests. Important contributions to the field investigations were made by D. Georgeson, R. Truy, and J. M. Wool. The Department of Water Resources provided drilling personnel and equipment, performed the field seismic surveys, supervised the borings and sampling operations, and performed all standard laboratory tests and a part of the dynamic testing on the samples at its Blyte laboratories near Sacramento. Major contributions to the investigation were made by C. J. Cortright, P. M. Schwartz, J. W. Keyser, W. D. Hammond, R. E. Stephenson, A. A. Coluzzi, W. W. Peak, W. D. Pedersen, J. M. Parsons, G. Heyes, R. F. Laird, J. S. Nelson, W. Hogue, and J. Miller. The National Science Foundation, through research grants to the University of California at Berkeley and Los Angeles, supported the analytical studies, part of the dynamic testing, and the overall evaluation of the results obtained. Contributions to these studies were made by J. Fitton, B. D. Adams, A. Albais, and G. Lefebvre.

The contributions of the sponsoring agencies and of all the engineers involved are gratefully acknowledged. Special appreciation is due to C. J. Thiel of the National Science Foundation, C. J. Cortright of the Division of Safety of Dams.
and R. V. Pilips of the Los Angeles Department of Water and Power, without whose foresight the detailed study reported in the preceding pages might never have been possible.

APPENDIX.—REFERENCES


