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[a]Discussion period closed for this paper. Any other discussion received during this discussion period will be published in subsequent Journals.

TECHNICAL NOTES

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JOURNAL OF THE
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THE SLIDES IN THE SAN FERNANDO DAMS DURING THE EARTHQUAKE OF FEBRUARY 9, 1971

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INTRODUCTION

One of the major effects of the San Fernando earthquake of February 9, 1971 was a major slide in the upstream slope of the Lower San Fernando Dam. This embankment dam, about 140 ft (43 m) high at its maximum section, provided a reservoir capacity of about 20,000 acre-ft (25,000,000 m^3) and at the time of the earthquake the water level in the reservoir was about 35 ft (11 m) below the crest. The slide movement resulting from the earthquake shaking involved both the upstream slope and the upper part of the downstream slope, leaving about 5 ft (1.5 m) of freeboard in a very precarious position. The upper part of the remaining embankment contained several large longitudinal cracks and the upstream face of the slide scarp was almost vertical. It seemed likely that further sliding might occur, especially if the embankment was subjected to a severe after-shock. Accordingly, an order was immediately issued to evacuate some 80,000 people living downstream of the dam until the level of the reservoir could be lowered to a safe elevation. This was accomplished in a period of 4 days. However, the margin by which a major disaster was averted was uncomfortably small.

Somewhat less serious, but of major importance in its own right, was a downstream slide movement in the Upper San Fernando Dam, about 80 ft (24 m) high and forming a reservoir of about 1,850 acre-ft (2,300,000 m^3). The two dams form the major part of the Van Norman Lake complex; the total

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complex involving one additional dam for water storage, some smaller dikes forming storm diversion structures, debris collection basins, and several miles of channels and artificially improved reservoir side slopes. An aerial photograph of the complex is shown in Fig. 1.

The downstream movement of the Upper Dam led to some cracking of the embankment, opening up of joints in the outlet conduit throuth the embankment, and the formation of a sink-hole along the line of the conduit due to erosion through the joints. However, although the crest moved downstream about 5 ft (1.5 m) and settled about 3 ft (0.9 m) there was no breach, resulting in loss of water from the reservoir. If this had not been the case and water from

FIG. 1.—Aerial View of Van Norman Lake Complex Taken after Earthquake (Department of Water Resources)

the Upper Van Norman Lake had been released, overtopping of the Lower Dam would have ensued.

The near-catastrophe resulting from the earthquake-induced slides in the Upper and Lower San Fernando Dams immediately raised a number of questions concerning the adequacy of earth dam design criteria to protect the public against failures resulting from earthquake shaking. Both of the affected dams were old structures constructed in the period from 1915-1925, and both were hydraulic fill construction. However, the safety of the Lower Dam had been evaluated using current design procedures and seismic criteria as recently as 1966 and it had been considered to be adequately resistant to earthquake effects. Accordingly, the performance of the dams raised the following major questions:

(1) Were the slide movements initiated primarily in the embankment soils or in the foundation soils; (2) what was the mechanics of sliding and were the slides the result of soil liquefaction or were they the result of the more conventional sliding block type of movement; (3) could the slides have been anticipated using available analytical or design procedures; and (4) are new criteria required for evaluating the safety of earth dams to earthquake effects? To provide answers to these questions, a major study of the slide movements was undertaken (Seed, et al., 1973). An abbreviated description of the study and the conclusions resulting from it are described herein.

History of Dams and Conditions Prior to Earthquake

Geology and Foundation Conditions.—The embankment of the Lower San Fernando Dam in the channel section and the lower portions of the abutments rests on recent alluvium consisting of stiff clay with lenses of sand and gravel. This alluvium attains a maximum thickness of about 35 ft (11 m) beneath the dam.

Underlying the alluvium and forming the upper parts of the abutments for the dam are shales, siltstones, and sandstones (State of California Department of Water Resources, 1971). The east or left abutment consists primarily of shales and siltstones of Upper Miocine age, the upper 30 ft-50 ft (9 m-15 m) being weathered to varying degrees and containing numerous gypsum-filled seams along joints, fractures, and bedding planes. Solution of this gypsum by reservoir waters throughout the years is believed to have been responsible for excessive seepage through the left abutment. Extensive grouting of the abutment in 1964 significantly ameliorated this condition.

Forming the right abutment and underlying the westerly part of the foundation alluvium is a massive friable sandstone of the Pico Formation (Middle Pliocene).

Exploration subsequent to the earthquake in an old borrow area on the west abutment revealed several faults about 100 ft (3 m) downstream from the axis of the dam. These faults cut the overlying alluvium and showed vertical offsets of 2 ft-5 ft (0.6 m-1.5 m). They were not traceable laterally, but possibly extended under the dam. There is no indication that any movement occurred on any of these faults during the February 9th earthquake.

The embankment of the Upper Dam is founded on deposits of recent alluvium, consisting of stiff clays and clayey gravels about 50 ft-60 ft (15 m-18 m) in thickness. Underlying the alluvium and forming the abutments of the dam are poorly-cemented conglomeritic sandstone and coarse-grained sandstone of the Saugus Formation (Lower Pleistocene).

Construction of Lower San Fernando Dam.—Construction of the Lower San Fernando Dam (Fig. 2) began in 1912. The foundation alluvium was not stripped off prior to placing the embankment fill. However, there are reported to be three cutoff trenches through this alluvium that extend down into the bedrock, and which were backfilled with puddled clay. One of these trenches containing plastic clay was encountered in one of the recent exploratory drill holes.

Judging by the available early photographs, the embankment was constructed by first making a broad dike of wagon-dumped and rolled fill at both the upstream and downstream edges. The large central area between the dikes was then filled by standard hydraulic fill procedures. Unfortunately, the lateral and vertical

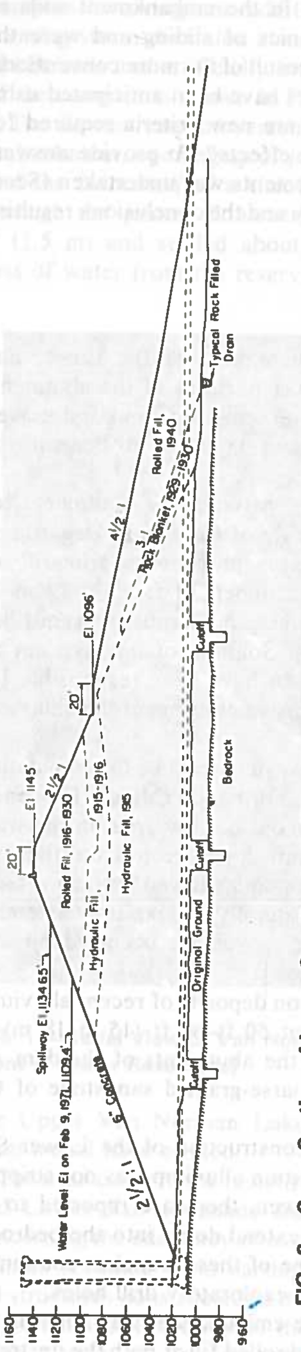


FIG. 2.—Cross Section of Lower San Fernando Dam (Los Angeles Department of Water and Power) (1 ft = 0.305 m)

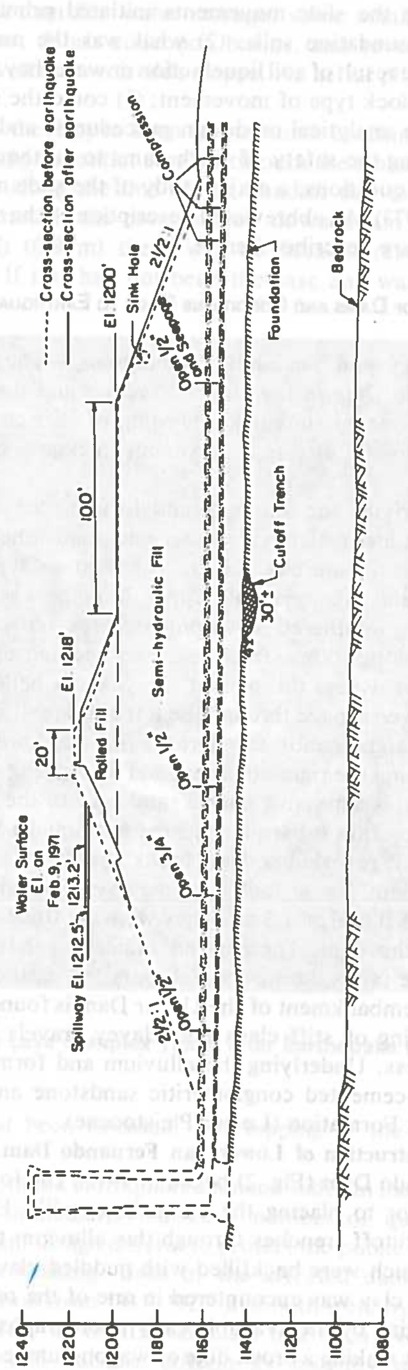


FIG. 3.—Cross Section of Upper San Fernando Dam (Los Angeles Department of Water and Power) (1 ft = 0.305 m)

extent of these dry fill dikes is unknown.

Between 1912 and 1915 the embankment was constructed to about El. 1,080 (329 m) at the axis and El. 1,090 (332 m) at the upstream and downstream edges [stream bed at the axis was approximately at El. 995 (303 m)] using material hydraulicked from the floor of the reservoir. Records indicate that the borrow area was then shifted and hydraulic construction was continued using ground-up shale from a borrow area on the hillside at the left end of the dam, until the dam was built to El. 1,097 (334 m) at the axis. Because the hydraulic fill process requires beaches and a slope toward the axis from both faces, it is probable that the top of the hydraulic fill section at the edges could be as high as El. 1,105 (337 m). In 1916-1917 the hydraulic fill section was capped by a rolled earthfill composed of shales from the east abutment. This fill was placed to about El. 1,118 (341 m) for a narrow width at the upstream side and El. 1,108 (338 m) across the remainder of the dam. In 1920 additional fill was placed to bring the upstream edge to El. 1,125 (343 m) (Los Angeles Department of Water and Power, 1929).

In 1924 the embankment was again raised. This time, rolled fill was placed to about El. 1,133 (345 m) along the upstream side and El. 1,118 (341 m) on the downstream side. The material used was a combination of heavy clay and gravel from a hill at the right end of the dam.

In 1929-1930 the dam was raised for the last time to El. 1,144.6 (349 m). A trench was excavated through all the previously placed rolled fill zones and into the hydraulic fill. All shale materials encountered were removed and the new fill was placed against what was reported to be a very plastic material. The shale material excavated from the core trench was mixed with gravelly material from borrow pits at the right end and upstream side of the dam and placed in a downstream toe addition. This is called a rock blanket in some of the reports; however, summary notes recorded in Ref. 4 (Los Angeles Department of Water and Power, 1929-1930) indicated it was not rock. This addition was placed on a 3:1 slope to El. 1,074.

In 1940 a final major modification was made with the construction of a rolled earth downstream toe addition terminating in a 20-ft (6-m) berm at El. 1,096 (334 m). This addition has a 4-1/2:1 slope but it steepens to 3-1/2:1 at the right end.

Thus, the dam can best be described as essentially a hydraulic fill embankment capped by a potpourri of wagon-dumped and rolled fills, founded on alluvium with three cutoff trenches to bedrock, and with a 20-ft (6-m) downstream berm at El. 1,096 (334 m). It has an upstream slope of 2-1/2:1, downstream slopes of 2-1/2:1 and 4-1/2/1, a height of 142 ft (43 m), a crest width of 20 ft (6 m), and a length of 2,080 ft (634 m). It was faced upstream with lightly reinforced concrete and had a 3-ft (0.9-m) high concrete parapet wall at the upstream edge of the crest. Altogether, about 3,300,000 cu yd (2,500,000 m^3) of embankment were used in construction to impound 20,500 acre-ft (25,300,000 m^3) of water.

Work done in 1929-1930 was during the period of an emergence of better moisture control and use of better compaction equipment. Light sheepsfoot tampers, either mounted on drums and pulled by crawler tractors or on rims mounted on the rear wheels of Fordson tractors, were used for compaction. The drum-mounted feet were reported to exert a pressure of about 71 psi (490 kN/m^3) on the soil. Efforts were also made to place the soil in controlled lifts and to add water by sprinkling. The additional fill added in 1940 was a well-controlled and well-compacted embankment. A cross section of the completed dam is presented in Fig. 2.

The dam was operated for many years with the reservoir peaking at its full design elevation of 1,134.6 ft (345.8) [approx 140 ft (43 m) above the old stream bed]. However, in 1966, following various engineering studies and reviews, the maximum operating reservoir level was reduced by 9.6 ft (3 m) to El. 1,125 (343 m).

Construction of Upper San Fernando Dam.—Like the Lower Dam, the Upper San Fernando Dam (Fig. 3) was also constructed directly on the alluvial soil.

Available cross sections of the dam show a cutoff trench extending to a depth of 4 ft (1.2 m) into alluvium for a width of 30 ft (9 m). This was probably intended only to cut off rodent holes and vegetation.

The main body of the dam was constructed by the semihydraulic fill method; the material being hauled from the borrow area to the edges of the embankment by wagons, dumped into the pond, and dispersed by monitors working from floating barges. The semihydraulic fill portion was constructed to about El. 1,200 (366 m) in 1921 [stream bed elevation was about 1,150 ft (351 m)] by using about 500,000 cu yd (383,000 m^3) of material obtained from the valley floor. Although it was originally planned to be constructed to El. 1,238 (372 m) in 1922, the dam was instead raised to El. 1,218 (371 m) by placing some 50,000 cu yd (38,000 m^3) of compacted dry fill on the upstream side. The dry fill material was obtained from sidehill borrow, spread in thin layers, sprinkled, and wagon-rolled. The completed section of the dam has a 2.5:1 concrete-paved upstream slope, a crest 20 ft (6 m) wide, and a downstream slope of 2.5:1 with a 100-ft (30-m) berm at El. 1,200 (370 m). A cross section of the embankment is shown in Fig. 3. Reservoir capacity is about 1,850 acre-ft (2,300,000 m^3).

Seismic Stability Investigations Prior to Earthquake

In a general review of the seismic stability of earth dams throughout California that was conducted in 1966, the earthquake resistance of the Lower San Fernando Dam was investigated by means of a conventional analysis procedure using a seismic coefficient of 0.15. This value was recommended by a consulting board appointed by the Los Angeles Department of Water and Power, based on the known and expected seismicity of the region.

The strengths of the soils comprising the embankment were determined by means of drained direct shear and triaxial compression tests on undisturbed samples. The rate of loading in these tests may have been too fast for full drainage to occur, but the data were interpreted conservatively to provide strength parameters for analysis purposes.

Stability computations were made using the conventional method of slices for the combined effects of: (1) An earthquake represented by a seismic coefficient of 0.15; and (2) a partial drawdown of the reservoir level from El. 1,125-El. 1,110 (343 m-340 m). These computations showed a minimum factor of safety of 1.05.

Based on the results of these studies, it was concluded that because the method of analysis was based on conservative strength values and force applications in keeping with conventional practice, the dam was safe against any anticipated ground motion if the water level was not allowed to exceed El. 1,125 (343 m). The reservoir was operated with this restriction during the following years.

Effects of Earthquake on Dams

Earthquake.—The San Fernando earthquake occurred at 6:00 a.m. local time and has been assigned a Richter magnitude of 6.6. The focal depth was about 8 miles (13 km). The earthquake was accompanied by thrust faulting in which the north block moved relatively up and over the south block at an angle of about 45° leaving a surface scarp. In some areas near the eastern end of this scarp, the relative upward movement amounted to more than 4 ft (1.2 m). The magnitude of these movements diminished and the visible surface breaks became discontinuous toward the western end of the scarp. However, features resembling a fault break were traced to the eastern edge of the Lower Van Norman reservoir.

The strong motion shaking produced by this earthquake was recorded at a number of locations within the area of high-intensity shaking. One of the most interesting records was that obtained at a station some 55 ft (18 m) above the left abutment of the 365-ft (111-m) high Pacoima Dam, located about 5 miles (8 km) south of the epicenter and 3 miles (4.8 km) east of the San Fernando Dams. The two horisontal components of ground motion at this station showed maximum accelerations of about 1.25g and the vertical component showed a maximum acceleration of 0.72g. However, it appears that the very high acceleration peaks in this record were probably due to the peculiar topographic conditions of the recording station and that substantially smaller accelerations would have been recorded near the base of the dam. An argument in favor of this is the lack of damage to the caretaker's house located in the valley a few hundred feet from the dam (the house did not even lose its chimney during the earthquake). However, even allowing for the amplifying effects of the local topography, it seems likely that the maximum acceleration in the vicinity of the Pacoima Dam was about 0.75g.

A second set of instrumental records of special interest in the study of the San Fernando Dams were those obtained on two seismoscopes; one located on the east abutment of the Lower Dam and one on the crest of the Lower Dam. The location of these instruments relative to the dam immediately after the earthquake is shown in Fig. 4. The instrument located on the crest of the dam was carried into the reservoir by the slide, and became submerged below the surface of the reservoir water. Fortunately, it was not damaged, and the instrument was recovered after the water had subsided.

Photographs of the traces made by the seismoscopes located on the abutment and crest of the Lower Dam are shown in Fig. 5. It may be seen that the motion of the rock abutment of the dam appears to have no preferred direction wheras the motion of the crest appears to be strongest in the traverse direction. However, detailed study shows that the seismoscope on the crest recorded a duration of motion somewhat comparable to the recorded on the abutment, thus indicating that the slide probably did not occur until near or just after the end of the stronger earthquake motions.

A detailed study and an ingenious interpretation of the trace of the seismoscope

FIG. 4.—Aerial View of Lower San Fernando Dam after Earthquake of February 9, 1971 (Arrows Indicate Seismoscope Sites) (United States Geological Survey)

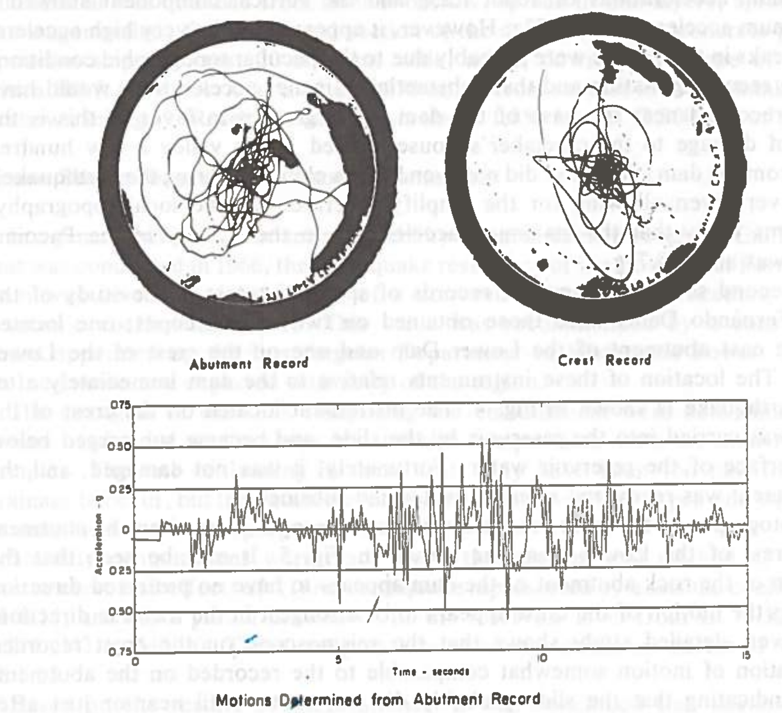


FIG. 5.—Seismoscope Records at Lower San Fernando Dam during Earthquake of February 9, 1971

record from the east abutment has been made by Scott (1973), who observed that some of the small regular waves on the trace were a peculiarity of the instrument and not of the earthquake. These waves provided a time scale to the record, from which it was possible to convert the trace on the seismoscope into a time history of acceleration, as shown in the lower part of Fig. 5. Some uncertainties developed where the instrument reached its maximum travel and bounced against its support, and where in several instances, the pen went off scale. Allowing for the uncertainties involved, Scott suggested that the peak acceleration developed should probably be no greater than 0.55g-0.6g. An independent analysis by Duke et al., indicated a maximum acceleration in the rock abutment at the damsite of about 0.5g with no significant amplification or attenuation between the rock and the crest of the dam.

Unfortunately, the seismoscope record from the crest of the Lower Dam has not yet been converted into an accelerogram.

Earthquake Effects on Upper Dam

An aerial view of the Upper Dam taken about 12 days after the earthquake is shown in Fig. 6. Severe longitudinal cracks are clearly evident running almost the full length of the dam on the upstream slope. At the time of the earthquake, the water level in the reservoir was above these cracks so that they were only visible after the reservoir level had been drawn down. These cracks resulted from a general downstream movement and settlement of the top portion of the dam with respect to the foundation. Subsequent surveys showed that at the center line, the crest moved downstream about 5 ft (1.5 m) and settled vertically about 3 ft (0.9 m) (Fig. 3).

A closeup view from the ground along the crest of the dam showing the bowing of the parapet wall resulting from the downstream movements is shown in Fig. 7 and a closeup view of the upstream cracks is presented in Fig. 8. These cracks appear to be multiple shear scarps at the outer edge of the fill.

At the downstream toe of the dam, a 2-ft (0.6-m) high pressure ridge developed and a 3-ft (0.9-m) diam concrete manhole near the east abutment was tilted and sheared noticeably in the downstream direction. These movements clearly show that the entire upper part of the dam participated in the movement downstream.

Shortly after the earthquake, the interior of the outlet conduit was surveyed to determine the extent of damage. The results of this survey are shown in the cross section of the dam in Fig. 3. In the central and upstream portion, there were several 1/2 in.-3/4 in. (13 mm-19 mm) cracks indicating extension movement in this zone. Near the toe of the dam there were compression failures in the conduit. The nature and direction of the movements at the conduit as indicated by these observations were generally the same as those observed from the surface. However, the magnitude of the cumulative movements at the conduit level was considerably less than the observed 5-ft (1.5-m) movement at the crest, indicating that the major movements either occurred within the fill above the conduit, or that the fill slipped along the outside edge of the conduit. The first possibility is considered to be the most probable.

Other surface features indicating the nature of damage within the dam included a large sinkhole above the downstream portion of the outlet conduit. This extended

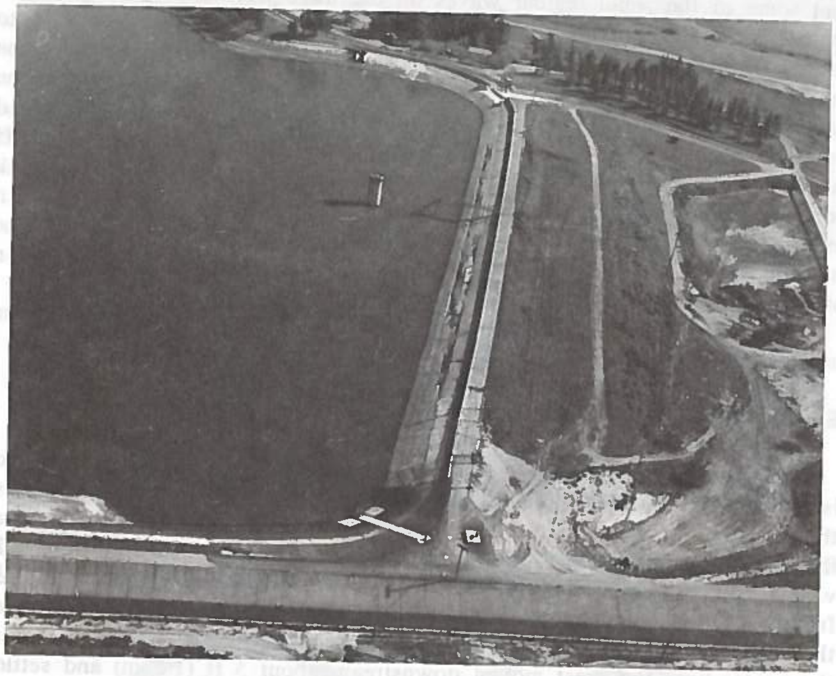


FIG. 6.—View of Upper San Fernando Dam after Earthquake (Department of Water Resources, February 21, 1971)

FIG. 7.—Deformation of Parapet Wall Resulting from Downstream Movement of Embankment

to the surface and its location is indicated on Fig. 3 almost directly above an open crack in the conduit. It was apparently formed by seepage and erosion through the cracks in the conduit during or immediately following the earthquake.

FIG. 8.—Slide Scarps on Upstream Face of Embankment

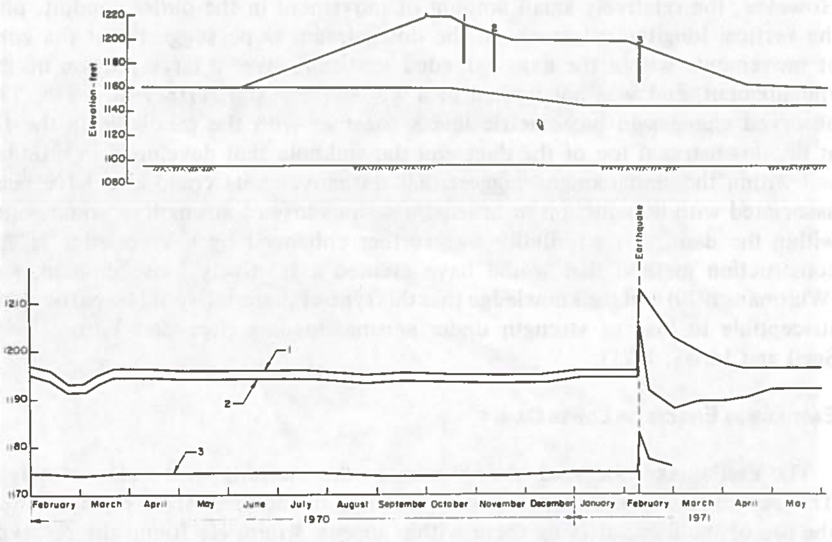


FIG. 9.—Changes in Water Level in Piezometers Following Earthquake, Upper San Fernando Dam (1 ft = 0.305 m)

In the area below the downstream toe of the dam, several sandboils were formed. The soil in this area consisted of about 8 ft (2.4 m) of loose silty sandy fill overlying alluvium. The origin, date, and method of placing this fill is unknown.

In addition to the transverse movements previously described, there was also some relative longitudinal movement of the embankment. However, the total amount of relative longitudinal movement appeared to be less than 2 ft (0.6 m), and it was therefore considerably less significant than the downstream movements.

Of special interest in studying the causes of the movements described were the observations of water level in three piezometers that had been installed in the dam for surveillance purposes prior to the earthquake. The locations of these piezometers in the cross section and the water-level changes following the earthquake are shown in Fig. 9. The effect of the shaking was to cause an immediate increase in pore-water pressure in the embankment, which slowly dissipated with time following the earthquake. As may be seen from Fig. 9, the recorded changes in pore pressure ranged from 8.5 ft–17 ft (2.6 m–5.2 m) of water. However, the increases for piezometers 1 and 2 near the center of the embankment were so large that water spilled over the tops of the well casings and the maximum values could therefore not be measured. Furthermore, since the first observations on these piezometers were made about 24 hr after the earthquake occurred, the actual increase in pore-water pressure in piezometer 3 is likely to be substantially higher than that shown.

The field observations at the Upper Dam suggest that the movements were due to increases in pore-water pressure and a corresponding weakening of the soil within a large portion of the dam. Near the top of the upstream face these movements appeared to be concentrated in two or three well-defined slip surfaces. However, the relatively small amount of movement in the outlet conduit, plus the vertical longitudinal crack on the downstream slope suggest that the zone of movements within the dam extended vertically over a large portion of the embankment, and was not limited to a well-defined slip surface at depth. The observed changes in piezometric levels together with the sandboils in the fill at the downstream toe of the dam and the sinkhole that developed in erodible soil within the embankment suggest that the movements could also have been associated with liquefaction or at least a serious loss of strength of some zones within the dam. this possibility was further enhanced by a knowledge of the construction method that would have created a relatively loose embankment (Whitman, 1970) and the knowledge that this type of material would be particularly susceptible to loss of strength under seismic loading (Lee and Fitton, 1969; Seed and Idriss, 1971).

Earthquake Effects on Lower Dam

The earthquake occurred at 6:00 a.m. in the morning, just before daylight. The caretaker of the dam immediately walked to the crest from his home near the toe of the dam, arriving there within approx 5 min. He found the reservoir to be perfectly quiet with no waves or sloshing action. Subsequent inspection around the shoreline gave no indication that there had been any abnormal waves or seiches resulting from the earthquake or the subsurface landslide.

At the lowest point of the embankment, the water level in the reservoir was only about 5 ft (1.5 m) below the sharp crest of the remaining near-vertical scarp. Numerous large longitudinal cracks had developed behind the scarp at lower elevations offering a potential for still further sliding that could reduce

FIG. 10.—Longitudinal Cracks in Remaining Portion of Embankment, Lower San Fernando Dam

FIG. 11.—Slide Damage to Lower San Fernando Dam (Department of Water Resources, February 22, 1971)

or eliminate the meager precarious freeboard that remained. Some of these cracks are shown in Fig. 10.

Sandbags were immdeiately rushed to the site and used to build up and reinforce the lowest area. Also, as a precautionary measure, all of the 80,000 persons living within a 2-mile by 12-mile (3,2 km by 19 km) rectangular area below the dam were immediately evacuated and dept out of the area for a period of 4 days while the water level in the reservoir was lowered to a safe elevation.

Since mid-1966, the lower San Fernando Reservoir had been operated with the maximum water level restricted to El. 1,125 (343 m), about 10 ft (3 m) below the spillway crest. At the time of the earthquake, the water surface elevation was at about El. 1,109 (338 m) some 25 ft (7.6 m) below the spillway and about 35 ft (11 m) below the crest of the dam. The reservoir was storing about 11,000 acre-ft (13,600,000 m^3) of water compared to its maximum design capacity of 20,500 acre-ft (25,300,000 m^3). Water was flowing into the reservoir at about 474 cfs (13.3 m^3/s), and flowing out through the two outlet towers at about 390 cfs (11 m^3/s).

As soon as possible following the quake, all the inflow to the reservoir was turned off and the outflow was increased to the maximum possible rate. Fortunately, the outlet tower near the west abutment remained standing and was not damaged during the earthquake although the walkway leading from the crest of the dam to the tower was broken during the slide. However, the outlet tower located near the center of the dam had broken off during the earthquake and was not visible above the reservoir water level. It was later determined that this tower was broken off about 20 ft (6 m) above the base and had fallen over in an upstream orientation. The broken section of the tower was covered by the slide material thereby restricting the flow through this outlet to approx 100 cfs (2.8 m^3/s). However, within a short period of time the entrance to the tower had cleared sufficiently by erosion and the outlet was quite effective in removing water from the reservoir.

In addition to removal of water through the two outlet towers, some water was also drawn off through three 12-in. (300 mm) blow-off valves reaching into the unpaved channel downstream from the dam. The United States Army Corps of Engineers also provided additional capacity by installing pumps on the shoreline with a total capacity of about 70 cfs (2 m^3/s). During the period when all available outlets were being used, the maximum outflow from the Lower Van Norman Reservoir was about 700 cfs (20 m^3/s). Most of this water was taken into the city's water supply system and other storage reservoirs.

During the first few days following the earthquake, the water level in the reservoir dropped at an average rate of about 4 ft/day (1.2 m/day). After about 4 days, the water level had been lowered sufficiently to eliminate the danger of a breech occuring in the remaining portion of the dam and the 80,000 residents living downstream were permitted to return to their homes.

Piezometers, located in the downstream portion of the embankment that was not destroyed by the movements, were read and other surveillance data were taken as quickly as possible following the earthquake. It was noted that seepage had increased for a short period and several seepage flows became turbid at first but then cleared within 36 hr after the earthquake. The water levels in the piezometers showed an initial rise but then later returned to normal and slowly decreased as the water level in the reservoir was lowered. The recorded

water pressure increases for Well 37 in the foundation soil near the downstream toe of the embankment and for Well 64-J with a tip located near the downstream toe of the original embankment were approx 5 ft and 3 ft (1.5 m and 0.9 m), respectively. The actual increases in water pressure even in these locations are likely to have exceeded these values because no readings were taken until about 6 hr after the shaking had stopped. Pore-water pressure changes in the upstream shell of the embankment are likely to have been much greater.

As the reservoir drained, the full extent of damage to the dam became increasingly apparent. A photograph of the dam taken about 13 days following the earthquake is shown in Fig. 11. By this time the water level had been lowered about 31 ft (9.5 m). Much of the slide debris was then visible. The original upstream sloping face of the dam was paved with concrete and sloped at 2-1/2 horisontal to 1 vertical. As shown in Fig. 11, this upstream slope had moved out horizontally into the reservoir, dropped considerably in elevation, and was lying almost horizontally.

The surface characteristics of the slide debris consisted of a series of steps and scarps indicating multiple shear zones within the fill. Slide scarps were apparent, sloping in both the upstream and downstream directions, and detailed observations showed that large blocks of material, more or less intact, had moved a considerable distance out into the reservoir area. In fact, the final topography of the slide debris had the same general form of blocks, grabens, and wedges that characterized landslides in Anchorage during the 1964 Alaska earthquake, and which were shown to be a result of deep-seated liquefaction (Seed and Wilson, 1967).

Finally, after the water had been completely removed from the reservoir, it was possible to inspect other portions of the slide near the toe of the debris. Sand boils such as those shown in Fig. 12 were found in some of the depressions or grabens in the lower portions of the slide. A view of the extreme toe of the slide is shown in Fig. 13. The lighter-colored material on which the two men are standing is the toe of the main embankment soil. It is a fine silty sand that appears to have flowed out over the darker-colored silts that had collected in the bottom of the reservoir during the 60 yr of operation. The toe of the slide did not take the form of a pressure ridge that is typical of the more conventional type of landslide.

The field observations of the performance of the Lower Dam during the earthquake suggested that the slide was due to liquefaction of soil within the hydraulic fill portion of the embankment. Evidence of this was provided by:

1. The seismoscope record from the crest of the dam indicated that the slide developed after the earthquake had continued for some time when the ground motions had almost subsided following the period of strong shaking; thus, it did not occur when the induced stresses were high but rather under essentially static load conditions. This would only have been possible if there were a major loss of strength of some of the soil comprising the embankment.

2. The topography of the landslide debris observed after the water level had been drawn down was typical of the topography of the Turnagain Heights landslide during the 1964 Alaska earthquake, and of other landslides that are believed to have developed as a result of liquefaction well below the surface during earthquakes. These peculiar topographic features include grabens, multiple shear

surfaces many of which slope backwards into the debris, and blocks of material, more or less intact, which moved during the landslide.

3. Large increases in water pressure in the observation wells within the

FIG. 12.—Send Boils in Depression Scarps near Toe of Slide Debris, Lower San Fernando Dam

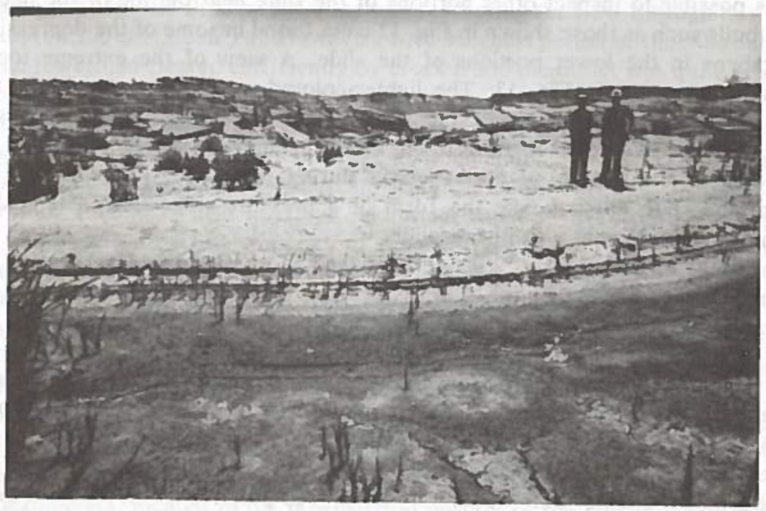


FIG. 13.—Toe of Slide Debris, Lower San Fernando Dam

embankment of the similarly constructed Upper Dam and significant increases in water pressure recorded near the downstream toe of the embankment of the Lower Dam.

4. The presence of sand boils that developed in the graben area near the toe of the slide where the overburden was shallow.

5. The nature of the toe of the slide that indicated that the material had simply flowed out over the existing reservoir sediments and suggesting that much of the material within the dam had been reduced to a fluid consistency.

It was surmised from these observations that probably a major zone of soil within the dam liquefied under the effect of the seismic loading during the later stages of the earthquake. However, this zone of liquefied soil was contained by an outer zone of stronger material that did not liquefy. Thus, the slide occurred when the liquefied zone was sufficiently extensive to produce an incipient failure condition; at this stage the outer shell began to move outwards and downwards, leading to a failure involving the crest of the embankment. Further studies to explore this possibility were therefore initiated.

Field Investigations of Slide Zone in Lower San Fernando Dam

Trenching.—To throw further light on the position of the failure zone and the mechanics of sliding, it was considered essential to cut a deep trench in

FIG. 14.—Test Strength through Portion of Slide Debris, Lower San Fernando Dam

the embankment to permit examination of failure zones or surfaces that developed during the sliding. Accordingly, a large trench was excavated into the slide debris on the upstream face at about the position of the central tower that had failed during the earthquake. A photograph of the western face of this trench is shown in Fig. 14. The maximum depth of the trench was about 60 ft (18 m) and the base was still about 20 ft (6 m) above the top of the alluvium.

The trench exposes most of the significant zones of fill as they appeared in their displaced positions following the slide. The upper part of the excavation exposes the rolled fill, under which is the relatively thin zone of hydraulically placed ground-up shale. These two zones are almost indistinguishable from each other, and even on close examination it was not possible to clearly define the boundary between them.

Immediately below the rolled and ground-shale fill is a zone of medium to coarse sand. In the field it had a bright yellow color and in the photograph it appears as a light-colored zone. It was about 4 ft (1.2 m) thick where it intersected the upstream face and tapered to almost zero thickness towards the left side of the photograph. In the extreme lower right-hand corner of the photograph more of this same zone of soil is seen. Apparently, during the slide a large block of soil slid down the present slope, twisted somewhat in the process, and came to rest with horizontal layers in an almost vertical position.

Below this marker bed of yellow sand are a large number of alternating light and dark layers of relatively undisturbed hydraulic fill. The fill in this zone closely resembled the undisturbed hydraulic fill exposed in trenches at the Upper Dam. The darker layers are silty to clayey material and the lighter layers are fine to coarse sands. These layers were more or less horizontal except toward the left-hand side of the photograph where they begin to curve noticeably downward; presumably they were pushed into this position by the slide movements.

Below and to the left of this stratified hydraulic fill was a large dark zone of homogeneous clay. This appears to have been the displaced clay core of the dam that had been pushed out into the sand layers of the hydraulic fill during the slide. The lower portions of the hhdraulic fill exposed in the trench to the right of this mass of clay showed considerable disturbance.

Near the bottom of the trench there was strong evidence of liquefaction of the sandy materials. A closeup view of this area is shown in Fig. 15. A vertical tongue or dyke of light-colored sand had extruded up through the darker soils, producing a configuration that could only have occurred if the sand dyke were moving as a fluid. Had there been a very shallow overburden of soil at this site, it is quite possible that the light-colored sand dyke would have reached the surface and formed a sand boil. It will be recalled that boils were found near the upstream toe of the debris where the overburden was very thin (Fig. 12).

When the trench section had been cut to a depth of about 60 ft (18 m), there were indications that the slope was becoming unstable and it was considered unsafe to carry the excavation any deeper. Work was therefore discontinued for a few weeks to give the soil a chance to drain and stabilize. Further excavation was then continued by means of a slit trench, which was extended about 16 ft (5 m) below the base of the large trench before it also had to be discontinued because of incipient failures in the side walls.

Most of this slit trench was cut through heavy clay material displaced from the core of the dam. However, the clay exposed in the walls of the slit trench contained a number of near-vertical cracks that were filled with sand. These sand-filled cracks could only have developed during the slide as a result of the sand liquefying and flowing into the fissures that developed in the distorting clay core.

Examination of the walls also showed pockets of sand about the size of baseballs, mixed within the homogeneous clay. It seems likely that these sand pockets were formed by the clay sliding over a bed of sand that had very little strength such that it could be picked up and mixed in the clay. This mixing occurred some 70 ft (21 m) or more below the original surface of the

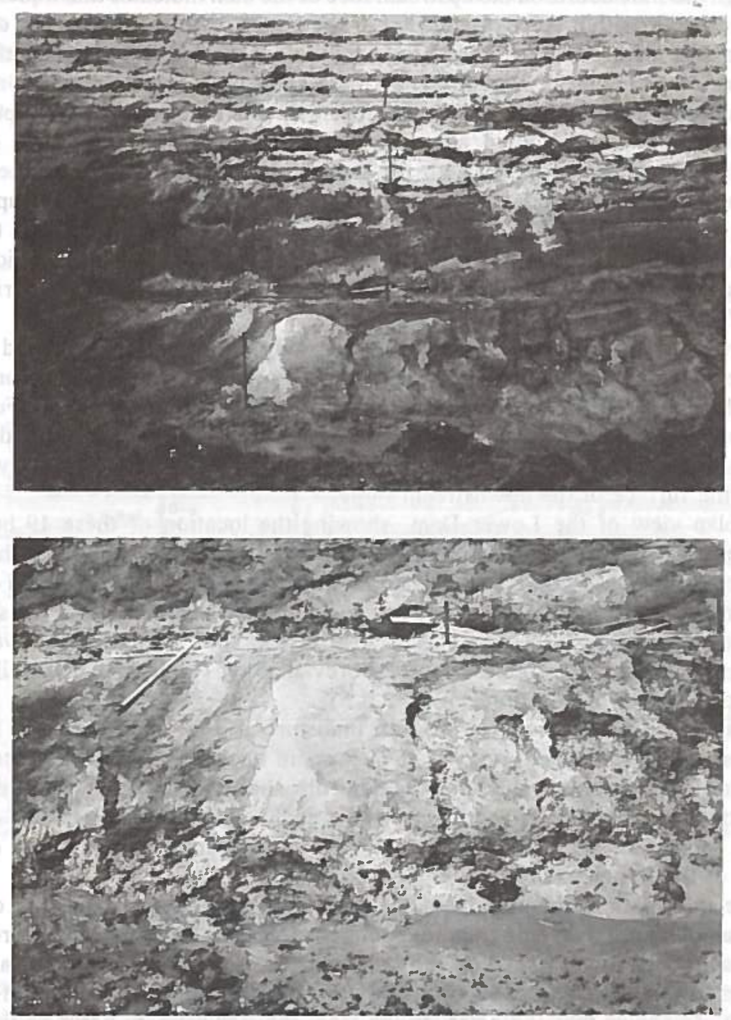


FIG. 15.—Enlarged Views at Bottom of Test Trench in Fig. 14

dam where, without liquefaction, the sand would have had considerable strength as a result of the heavy overburden pressure.

At the bottom of the slit trench a layer of water-bearing sand below the upper clay core material was found. Since the bottom of the trench was still about 4 ft (1.2 m) above the alluvium, this material was apparently the original hydraulic fill. The relative position of a thick layer of clay overlying sand is not at all typical of hydraulic fill in a normal condition and it was concluded that the clay must have been moved into its position over the sand sometime during the slide.

The information gained from examination of the soils exposed by the trenches through the slide debris on the upstream face of the dam indicates that liquefaction did not occur within the hydraulic fill in the upper regions towards the outside face of the dam. On the other hand, the evidence suggests that liquefaction did occur within the hydraulic sand fill in the lower central regions, and as a result, clay material from the central core was able to move out in an upstream direction over this liquefied zone. As the clay material moved, some of the liquefied sand became mixed with the lower portion of the clay; in some cases where cracks developed, some of the liquefied sand was extruded upwards filling cracks and forming sand dykes. On the upper surface of this tongue of clay that was pushed out into the sand, the clay merely pushed aside and through the layers of hydraulic fill, bending and shearing the previously horizontal stratifications.

Boring and Sampling.—Because the trenching could not be extended down to the alluvium foundation, it was necessary to explore the foundation soils and the remainder of the dam by a boring and sampling program. For this purpose, a total of nineteen 6-in. diam uncased auger holes were drilled along four sections across the dam from the surface of the slide debris as well as from the surface of the downstream slope.

A plan view of the Lower Dam, showing the location of these 19 holes is presented in Fig. 16. A typical cross section of the central portion of the dam where the trench section was located is shown in Fig. 17. The section shows in dashed outline the original profile of the dam, and in bold outline the surface profile of the dam as it was surveyed after the slide and after the water in the reservoir had been removed. An abbreviated log for each of the drill holes is also shown on the section.

A standard penetration test and an undisturbed 3-in. (76-mm) diam Shelby tube sample were taken every 5 ft (1.5 m) in each of the borings. Standard penetration resistance values for the soils are shown with the abbreviated drill logs on the sections. An outline of the trench section is also shown on the cross section in Fig. 17, together with the major zones of soil found on the walls of the trench excavation.

The information obtained from the trench excavation and from the drilling and sampling program along the four sections through the embankment provided the basic data for determining the nature of the soil within the dam, and for analyzing the movements that occurred as a result of the earthquake. Reference to Fig. 17 shows that the holes drilled from the downstream berm of the dam passed through soil that did not participate in the slide and was therefore probably not seriously disturbed as a result of the earthquake. For this reason, the undistributed Shelby tube samples from the downstream holes were used exclusively for laboratory tests to determine the characteristics of the soils comprising the embankment. On the other hand, the holes drilled through the central and upstream portions of the dam passed through materials that participated in the slide and were probably significantly disturbed because of the slide movements. Shelby tube samples from these areas were therefore used

only to identify the nature and extent of the slide movements that occurred.

Each of the Shelby tube samples from the central and upstream holes was split open in a longitudinal direction and carefully examined for evidence of disturbance during the slide. Photographs of the soil in two of these tubes are shown in Fig. 18. The soil in Fig. 18(a) consisted of stratified hydraulic fill overlying alluvium. The contact between these two zones is clearly evident in the photograph. The hydraulic fill consisted of numerous very thin layers of silty sand sandwiched between thicker layers of clay. All of the layers were plane and well defined, indicating that there had been virtually no disturbance of the soil in this zone.

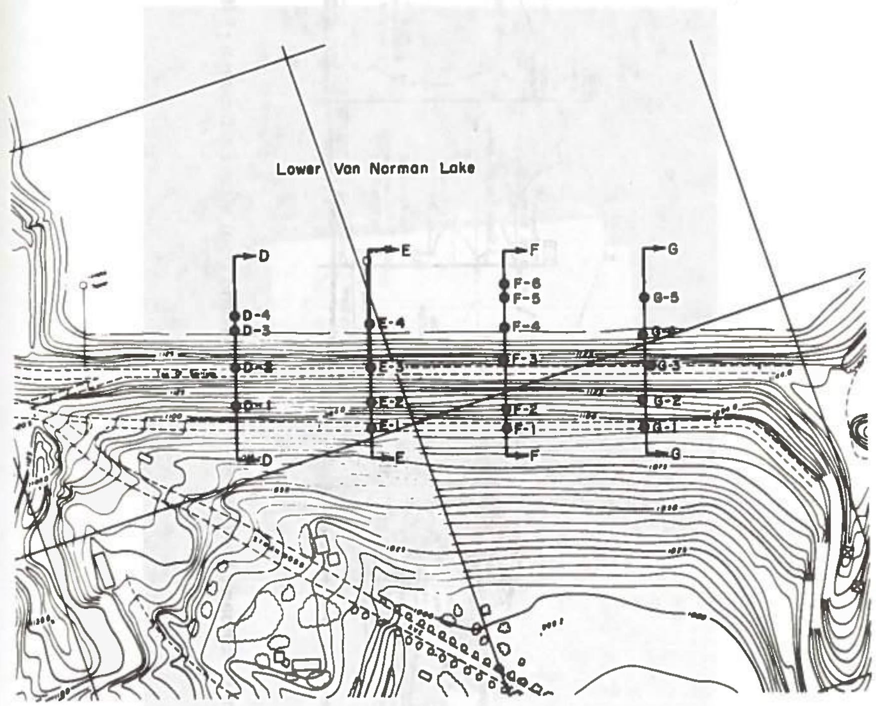


FIG. 16.—Plan of Lower San Fernando Dam

In contrast, the soil in the Shelby tube shown in Fig. 18(b) consisted of dark streaks of dark silty sand distorted and mixed in a zone of lighter-colored sald, indicating a large amount of disturbance. The pattern of coarse streaks within the sand shown in Fig. 18(b) could only have formed if the sand had been in a very weak or liquefied condition.

These photographs are typical of the soil that was found within the Shelby tubes taken from drill holes in the slide area through the upstream portion of the dam. Many of the samples showed no disturbance whatsoever while other samples showed a considerable amount of disturbance. The major zones of liquefaction disturbance as previously indicated were noted on the drill logs

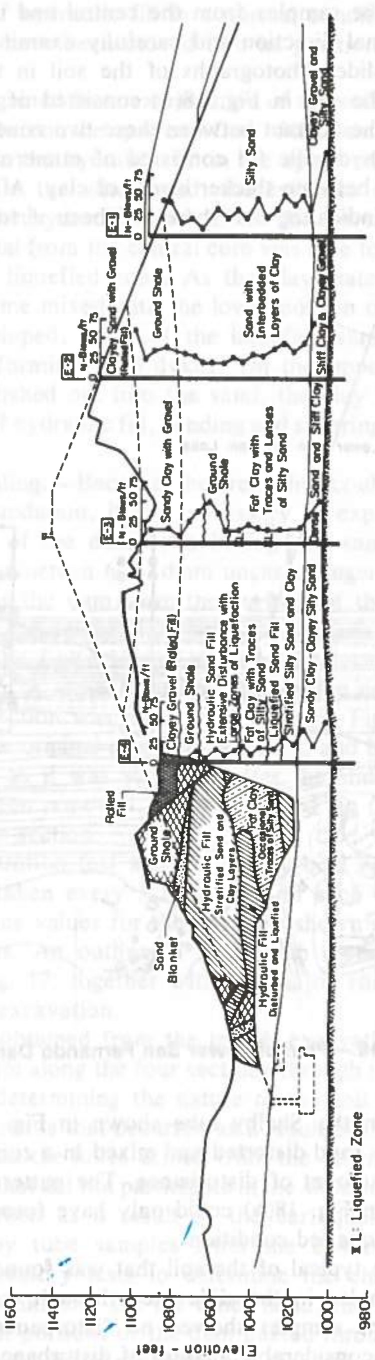


FIG. 17.—Cross Section of Embankment of Lower San Fernando Dam after Earthquake (1 ft = 0.305 m)

and shown on the sections through the dam (Fig. 17). As described in the following sections, these zones of disturbance played a key role in helping to reconstruct the movement that occurred as a result of the earthquake.

The detailed examination of the sample also led to the conclusion that the failure zone did not extend down into the foundation alluvium. Evidence leading to this conclusion included the following:

1. Although evidence of extensive disturbance and liquefaction could readily be recognized in many samples taken from the lower part of the hydraulic

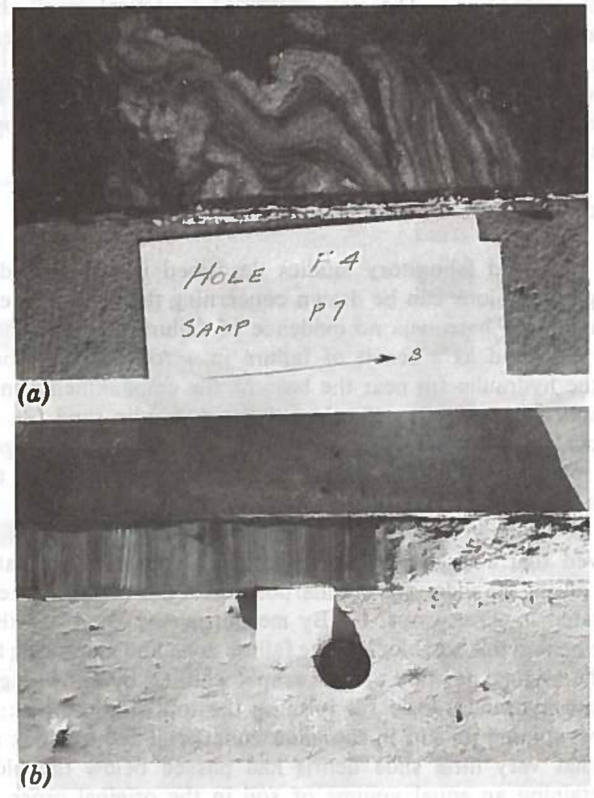


FIG. 18.—Examples of Shelby Tube Samples of Soil from Lower Dam

fill on the upstream side of the embankment [Fig. 18(*b*)], there was no indication of disturbance in any of the more than two hundred 7-in. (180-mm) long samples of the foundation alluvium.

2. Over a substantial part of the slide area, the foundation alluvium was overlain by a thin layer of undisturbed hydraulic fill indicating that the shear zone did not extend down to the alluvium [Fig. 18(*a*)].

3. In a number of locations, the contact between hydraulic fill and alluvium was observed in the samples recovered from the borings, thus determining the

elevation of the top of the alluvium after the slide. In all cases, this elevation was very close to that indicated on the construction drawings, indicating that no significant vertical movements had taken place in the alluvium.

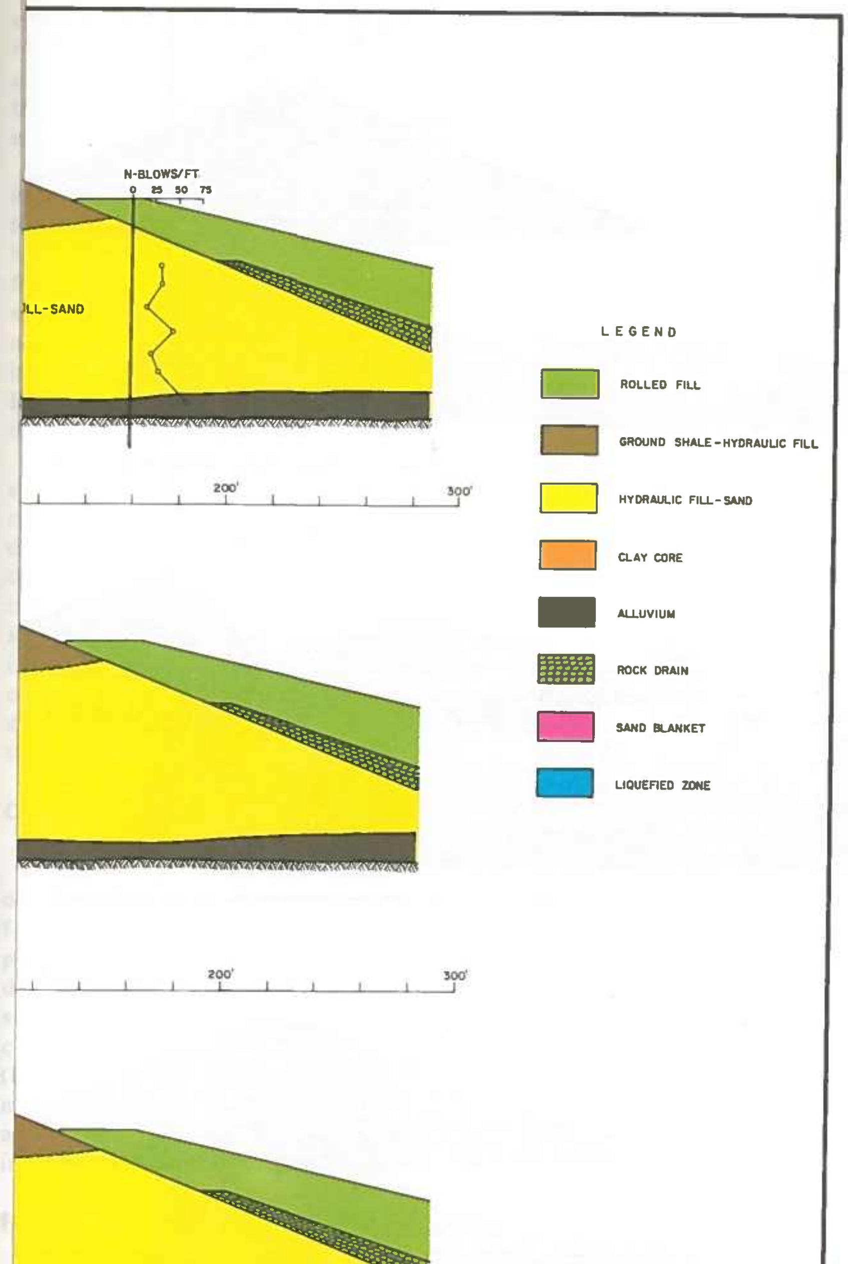
The Shelby tube samples from holes made in the downstream portion of the dam were shipped to the laboratory for testing purposes. There was very little evidence of soil disturbance in these samples. However, in a few rare cases, some indication of minor disturbance was noted and in these cases the samples were discarded and not used for testing. In one case, a 3-in. (76-mm) long seam of clay was found sandwiched between an upper and lower layer of medium to coarse sand. The clay contained a vertical crack about 1/2 in. (13 mm) wide which was completely filled with sand. Occasional vertical cracks such as this would seem to indicate that there may have been some zones of liquefaction within the sand even in the downstream portion of the dam. However, because of the large massive berm, the downstream slope remained stable and no slide movements developed.

Reconstruction of Mechanics of Sliding in Lower San Fernando Dam

From the field and laboratory studies described in the preceding section, the following conclusions can be drawn concerning the slide in the Lower San Fernando Dam: (1) There was no evidence of failure in the foundation soils; (2) the slide occurred as a result of failure in a zone of soil about 20 ft (6 m) thick in the hydraulic fill near the base of the embankment; and (3) failure was accompanied by some liquefaction of the hydraulic sand fill. Recognition of these facts, together with the possibility of identifying large blocks of undisturbed soil in the slide mass, made it possible to reconstruct the probable mechanics of sliding.

Consider, for example, the section E-E shown in colored Plate 1. The trench section showed that a large block of soil, block number 7 in Plate 1(*b*), had moved intact during the slide. The original position of this block in the embankment could be located in three ways: (1) By measuring the length of the pieces of concrete slab behind this soil block in the failure zone and measuring this distance down from the original position of the parapet wall; (2) by knowning the original position of the hydraulic shale fill forming the top of this block; and (3) by measuring the volume of soil in the slide zone behind this block and, on the assumption that very little slide debris had passed below the block, finding a position retaining an equal volume of soil in the original cross section. All three of these procedures gave similar initial locations for the block and it was determined to have moved from the location shown in Plate 1(*c*).

With the aid of the surface boundaries indicated by the original roadway across the top of the dam, the parapet wall and the concrete slab facing for the upstream slope, the internal boundaries between different zones of soil within the embankment that could be identified from the trench section and the boring logs, and some judgment, it was possible to recognize the large relatively undisturbed blocks of soil numbered 1-11 in Plate 1(*b*). Using techniques similar to those previously described, the approximate original positions of these blocks were then determined as shown in Plate 1(*c*). It was found that the blocks fitted together extremely well and formed the upper part of the original



LOWER SAN FERNANDO DAM
CROSS-SECTIONS
THROUGH EMBANKMENT
SECTION E-E
PLATE 1

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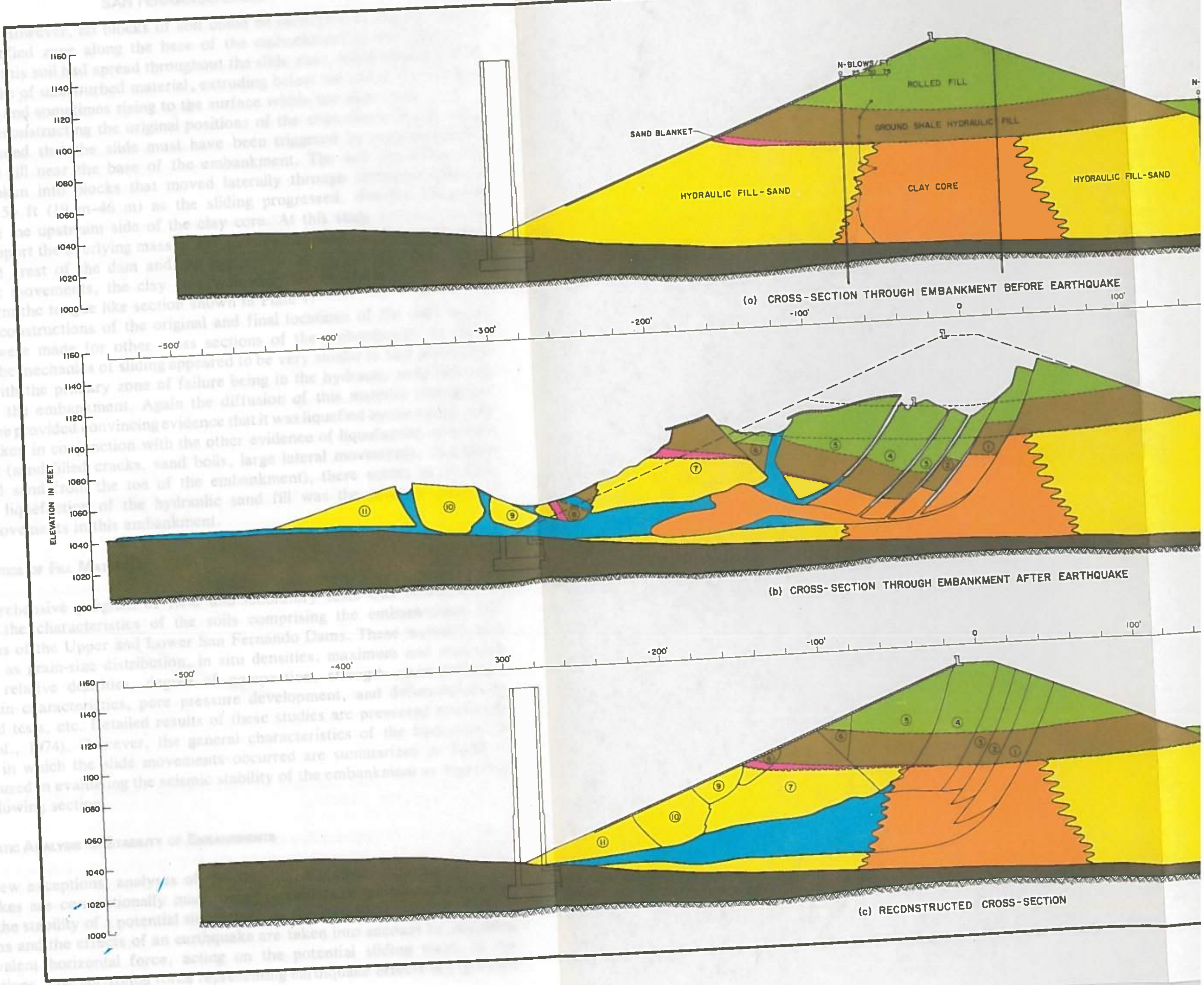
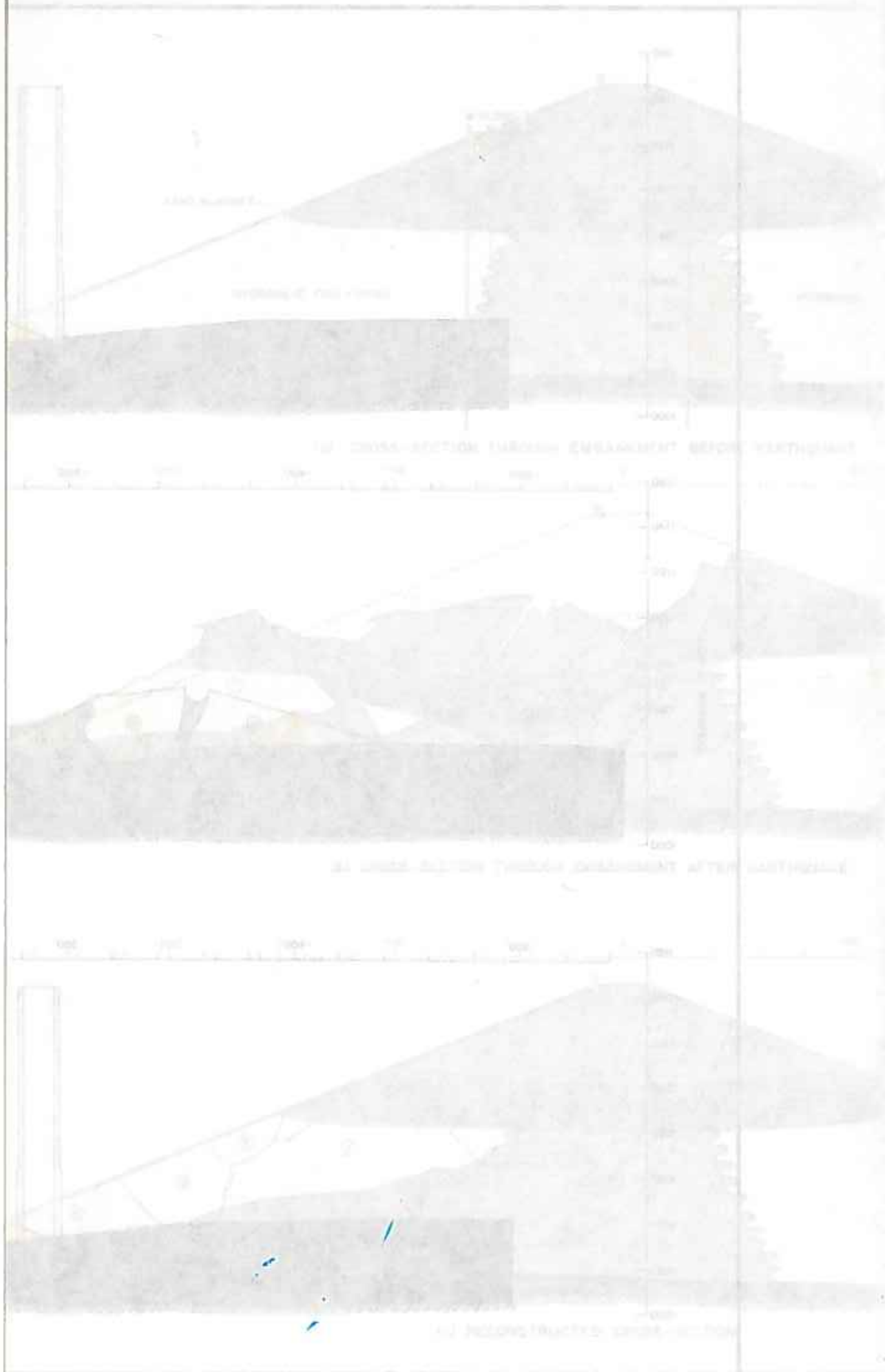


(a) CROSS-SECTION THROUGH EMBANKMENT BEFORE EARTHQUAKE

(b) CROSS-SECTION THROUGH EMBANKMENT AFTER EARTHQUAKE

(c) RECONSTRUCTED CROSS-SECTION



embankment. However, no blocks of soil could be identified as having moved from the liquefied zone along the base of the embankment in this figure. It appeared that this soil had spread throughout the slide zone, infiltrating between the large blocks of undisturbed material, extruding below the toe of the original embankment, and sometimes rising to the surface within the slide zone.

Thus, by reconstructing the original positions of the slide debris in this way, it was concluded that the slide must have been triggered by liquefaction of the hydraulic fill near the base of the embankment. The soil above this fill had then broken into blocks that moved laterally through distances varying from 30 ft-150 ft (19 m-46 m) as the sliding progressed, thereby removing support from the upstream side of the clay core. At this stage the core could no longer support the overlying mass of soil causing a secondary slide movement involving the crest of the dam and the upper part of the downstream slope. During these movements, the clay core was extruded by the overlying mass of soil to form the tongue like section shown in Plate 1.

Similar reconstructions of the original and final locations of the soils in the slide zone were made for other cross sections of the embankment. In these cases also, the mechanics of sliding appeared to be very similar to that previously described with the primary zone of failure being in the hydraulic sand fill near the base of the embankment. Again the diffusion of this material throughout the slide zone provided convincing evidence that it was liquefied by the earthquake shaking. Taken in conjunction with the other evidence of liquefaction observed in the field (sand-filled cracks, sand boils, large lateral movements, extrusion of liquefied sand from the toe of the embankment), there seems to be little doubt that liquefaction of the hydraulic sand fill was the primary cause of the slide movements in this embankment.

Characteristics of Fill Materials

A comprehensive program of field and laboratory tests was conducted to determine the characteristics of the soils comprising the embankments and foundations of the Upper and Lower San Fernando Dams. These included such properties as grain-size distribution, in situ densities, maximum and minimum densities, relative densities, degree of compaction, strength characteristics, stress-strain characteristics, pore pressure development, and deformations in cyclic load tests, etc. Detailed results of these studies are presented elsewhere (Lee, et al., 1974). However, the general characteristics of the hydraulic fill materials in which the slide movements occurred are summarized in Table 1, and were used in evaluating the seismic stability of the embankment as described in the following section.

Pseudostatic Analysis of Stability of Embankments

With few exceptions, analyses of the stability of embankment dams during earthquakes are conventionally made using pseudostatic methods. In this approach, the stability of a potential sliding mass is determined as for static loading conditions and the effects of an earthquake are taken into account by including an equivalent horizontal force, acting on the potential sliding mass, in the computations. The horizontal force representing earthquake effects is expressed as the product of the weight of the sliding mass under consideration and a seismic coefficient, k.

In United States practice, the value of the seismic coefficient is normally selected on the basis of the seismicity of the region in which the dam is to be constructed; values in California range from a lower limit of 0.05 to an upper limit of about 0.15. For the San Fernando dams in the 1971 earthquake, a value of 0.15 would therefore represent most normal design practice; this value was in fact used to evaluate the seismic stability of the Lower Dam in 1966.

TABLE 1.—Characteristics of Hydraulic Sand Fill

Characteristics (1)	Upper dam (2)	Lower dam (3)
50% size, D_{50}, in millimeters	0.05-0.8	0.05-1.0
Coefficient of uniformity	4-6	7-10
Dry unit weight, in pounds per cubic foot	90-115	95-110
Relative density, as a percentage	51-58	51-54
Degree of compaction based on standard AASHO test, as a percentage	92-98	95-100
Degree of compaction based on modified AASHO (1961) test, as a percentage	83-92	88-93
Effective stress strength parameters	$c' = 0$ $\phi' = 37°$	$c' = 0$ $\phi' = 37°$
Total stress strength parameters (average)	$c = 1{,}100$ psf $\phi = 24°$	$c = 2{,}040$ psf $\phi = 19°$

Note: 1 pcf = 0.157 kN/m³; 1 psf = 47.9 N/m³.

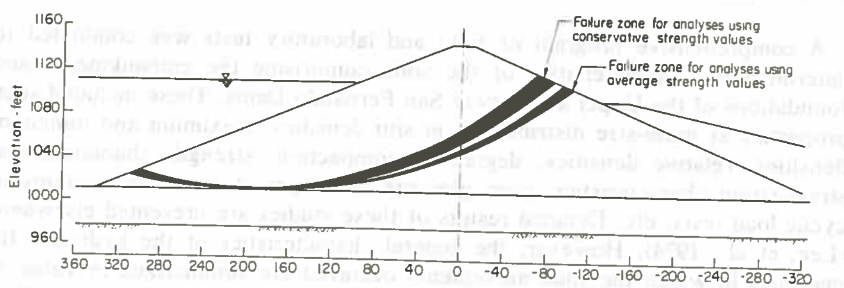


FIG. 19.—Failure Zones Indicated by Pseudostatic Stability Analyses, Lower San Fernando Dam (1 ft = 0.305 m)

To evaluate the applicability of this method of approach for determining the stability of the San Fernando dams in the 1971 earthquake, analyses have been made using the conventional method of slices to compute the factors of safety for the dams using a seismic coefficient of 0.15 and, since both dams presumably reached the point of incipient failure during the earthquake [the 5-ft (1.5-m) downstream movement of the Upper Dam is about the limit of tolerable movements], the values of seismic coefficient which would lead to a computed factor of safety of unity.

Lower San Fernando Dam.—Computation of the factor of safety against sliding of the upstream slope of the Lower Dam using a pseudostatic approach is complicated by the availability of different strength parameters for the embankment soils. For the short duration of the earthquake of February 9, 1971 it is apparent that strength parameters should correspond to undrained loading conditions. Thus, test data for consolidated-undrained triaxial compression tests provide an appropriate basis for analysis.

The data obtained in tests of this type on undisturbed samples of the hydraulic fill showed considerable scatter (Lee, et al., 1974), reflecting the variability of the soil in the embankment. Thus it was possible to interpret the data to determine a lower bound or conservative strength relationship for the shell material or to draw an average line through the data points. Strength parameters defining the envelope of failure for total stresses in consolidated-undrained tests on the shell material following both of these procedures were found to be: (1) Conservative—$c = 1{,}200$ psf (57.5 kN/m²), $\phi = 20°$; and (2) average—$c = 2{,}040$ psf (97.7 kN/m²), $\phi = 19°$. These strength parameters are somewhat higher than those used by the Los Angeles Department of Water and Power in the 1966 stability studies [$c = 720$ psf (34.5 kN/m²) and $\phi = 25°$] because of the even more conservative evaluation of the test data made at that time.

The undrained shear strength of the core material was determined from the shear strength/effective pressure ratio of 0.3 indicated by the test data for the clay in this part of the embankment. Shear strength values varied from about 1,700 psf (81.4 kN/m²) near the top of the core to about 2,600 (125 kN/m²) near the base. Test data for the foundation soils gave considerably higher strength parameters [$c = 1{,}560$ psf (74.7 kN/m²) and $\phi = 27°$] than those for the hydraulic fill.

In using the test data for stability computations, it is possible to use the strength parameters directly to determine the soil strength in the embankment or to convert the Mohr envelope into a relationship between the shear stress on the failure plane at failure τ_{ff}, and the normal stress on the failure plane at the end of the consolidation phase of the tests, σ_{fc}. Both procedures are apparently used in practice and analyses were made accordingly using both procedures in the present study.

The critical surfaces of sliding determined by the analyses are shown in Fig. 19 and the computed factors of safety for different seismic coefficients as determined by the conventional method of slices, are presented in Table 2. It may be seen that the computed locations of the most critical sliding surface are in reasonably good agreement with the position of the slide zone in the embankment, and that the analyses correctly indicate that failure would not extend into the foundation soils. However, for a seismic coefficient of 0.15, the computations indicate factors of safety ranging from about 1.22 for a conservative interpretation of the test data combined with a Mohr envelope method of data use, to about 1.61 for an average interpretation of the test data combined with the use of the τ_{ff} versus σ_{fc} strength relationship.

Because it has been common practice to interpret a computed factor of safety exceeding about 1.1 to be indicative of adequate seismic stability for an embankment, it is clear that the use of conventional pseudostatic analyses would not have predicted the failure that actually occurred.

However, as shown in Table 2, the use of seismic coefficient values ranging from about 0.22-0.34, depending on the method of data interpretation, would have led to a computed factor of safety of unity. Presumably, values of this order of magnitude would be required in pseudostatic analyses, even when other computational details are selected to minimize the computed factor of

TABLE 2.—Results of Pseudostatic Analyses of Stability

Embankment Strength Characteristics				
c, in pounds per square foot (1)	ϕ, in degrees (2)	Data form used in analysis (3)	Factor of safety for seismic coefficient, k = 0.15 (4)	Seismic coefficient, k, for factor of safety = 1 (5)
(*a*) Lower San Fernando Dam				
1,200	20	Mohr envelope	1.22	0.22
1,200	20	τ_{ff} versus σ_{fc}	1.37	0.25
2,040	19	Mohr envelope	1.47	0.29
2,040	19	τ_{ff} versus σ_{fc}	1.61	0.34
(*b*) Upper San Fernando Dam				
1,100	24	Mohr envelope	2.03	0.43
1,400	28	τ_{ff} versus σ_{fc}	2.49	0.55

Note: 1 pcf = 47.9 kN/m².

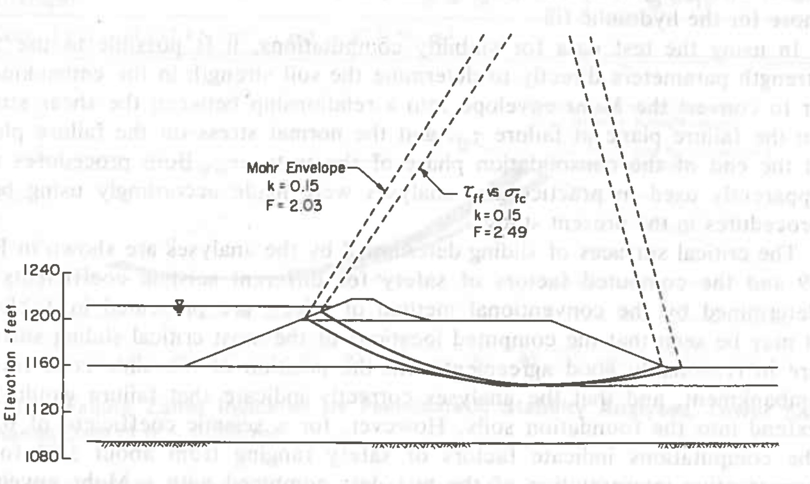


FIG. 20.—Results of Pseudostatic Analyses of Embankment Stability, Upper San Fernando Dam (1 ft = 0.305 m)

safety, to give factors of safety indicative of failure for earthquake motions comparable to those developed in the San Fernando area during the 1971 earthquake.

Upper San Fernando Dam.—Similar computations to those previously described

were made for the Upper San Fernando Dam. The strength parameters for consolidated-undrained triaxial compression tests for the embankment and foundation soils of this dam (Lee, et al., 1974) were: (1) Embankment—c = 1,100 psf (52.7 kN/m²), ϕ = 24°; and (2) foundation—c = 1,420 psf (68 kN/m²), ϕ = 32°. The results of the pseudostatic analyses are shown in Fig. 20 and Table 2. The computed position of the critical sliding surface (Fig. 20) is again in reasonable agreement with the observed behavior of the embankment during the earthquake, in that downstream movement occurred with the formation of scarps on the upstream face in approximately the same position as that indicated by the analysis. However, for a seismic coefficient of 0.15, the computed factor of safety was about 2.0 for soil strengths determined from a standard Mohr envelope and 2.5 for strengths determined from the τ_{ff} versus σ_{fc} form of data representation. In either case the analysis indicates an ample margin of safety against slide movements developing—a result in marked contrast to the actual behavior of the embankment.

To compute a factor of safety of 1.0 using the pseudostatic method of analysis, it would be necessary to use a seismic coefficient in the range of 0.43-0.55, depending on how the consolidated-undrained test data is utilized in the computation procedure. These values are substantially higher than those currently used for seismic design but they are apparently necessary for the pseudostatic method of analysis to correctly indicate the performance of the upper San Fernando Dam under the ground motion conditions that occurred in San Fernando in the 1971 earthquake.

Note that the analysis correctly indicates a lower factor of safety for downstream sliding than for upstream slides. Using the seismic coefficient of 0.43 which indicates a factor of safety of 1.0 for downstream sliding, the computed factor of safety against an upstream slide was about 1.12.

These results pose a number of difficult problems for design engineers. If seismic coefficients of the order of 0.25-0.5 are required to adequately assess the stability of these types of dams against shaking of the intensity developed in San Fernando, should values of comparable magnitude be used for similar dams that may be subjected to comparable levels of shaking—or even higher values for more severe shaking intensities? The use of such values would lead to much flatter slopes than have conventionally been used, probably unjustifiably in many cases, leading to unnecessary and prohibitive expense while providing little additional benefit in others; furthermore, considerable difficulty would be encountered in knowing under what conditions they might be justified as in the case of the Lower San Fernando Dam. In view of the many other limitations of pseudostatic analysis procedures (Seed, et al., 1969) it was considered desirable to explore the applicability of dynamic analysis procedures for evaluating the stability of the embankments. The results of such analyses are summarized in the following sections.

Dynamic Analyses of Stability of Lower Dam during San Fernando Earthquake

In view of the limitations of pseudostatic methods of analysis, procedures for dynamic analysis of embankment stability have been developed in recent years (Newmark, 1965; Seed, 1966). In dealing with saturated cohesionless

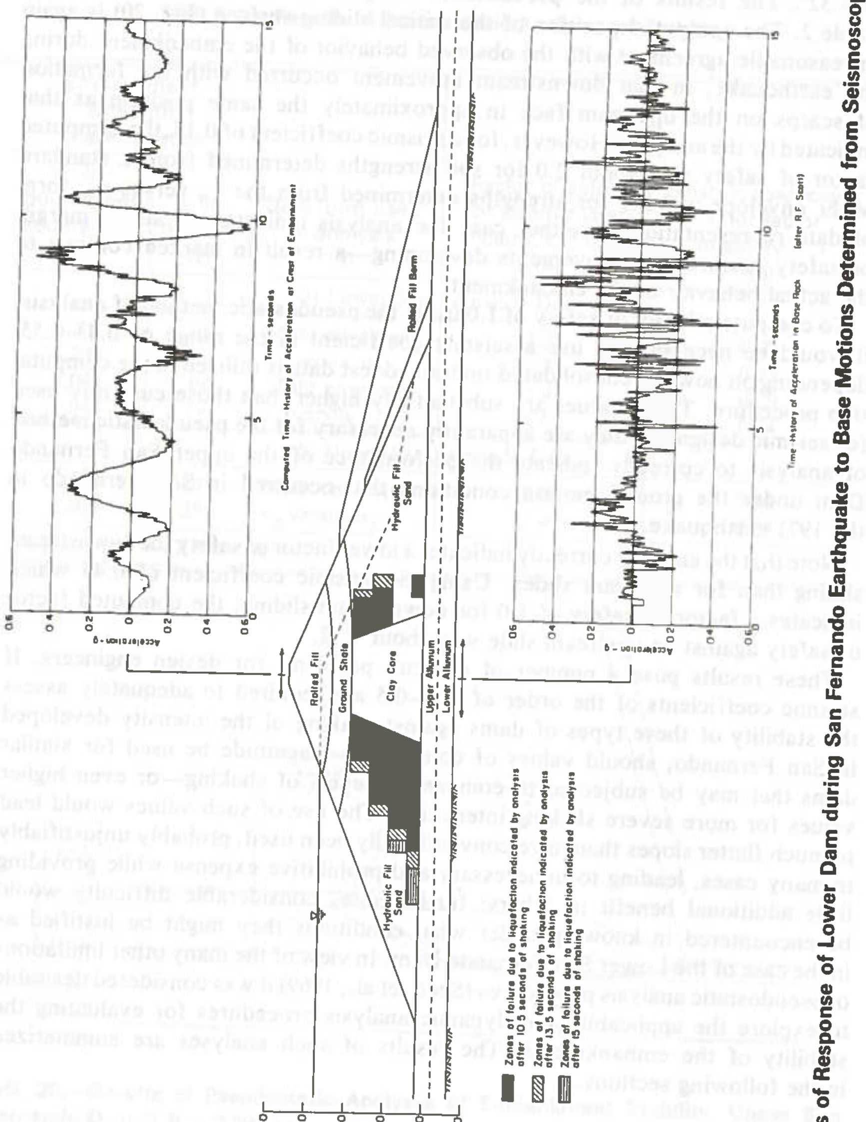


FIG. 21.—Analysis of Response of Lower Dam during San Fernando Earthquake to Base Motions Determined from Seismoscope Record (1 ft = 0.305 m)

materials for which pore pressures may vary during an earthquake, it has been found most convenient to utilize the procedure proposed by Seed involving the following steps:

1. Determine the initial stresses in the embankment before the earthquake.
2. Determine the characteristics of the motions developed in rock underlying the embankment and its soil foundation during the earthquake.
3. Evaluate the response of the embankment to the base rock excitation and compute the dynamic stresses induced in representative elements of the embankment.
4. By subjecting representative samples of soil to the combinations of pre-earthquake stress conditions and superimposed dynamic stresss applications, determine by test the effects of the earthquake-induced stresses on soil elements in the embankment. These effects will include any evidence of soil liquefaction and the magnitude of the deformations induced by the earthquake loading.

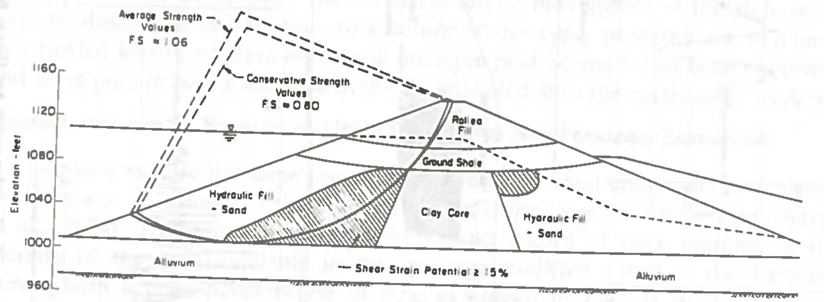


FIG. 22.—Analysis of Stability of Lower San Fernando Dam after Development of Zone of High Strain Potential (1 ft = 0.305 m)

5. From a knowledge of the deformations induced in individual soil elements in the embankment, evaluate the overall deformations and stability of the cross section.

This procedure has been found to provide a satisfactory evaluation of the failure of the Sheffield Dam during the Santa Barbara earthquake of 1925 (Seed, et al., 1969) and it has been used for design studies of a number of other embankment dams. Accordingly, it was adopted for analysis of the San Fernando dams in the 1971 earthquake.

The results of the analysis of the Lower Dam are summarized in Figs. 21 and 22. Fig. 21 shows the time history of base excitation determined by Scott from the seismoscope record and the computed time history of crest accelerations corresponding to the embankment response. Note that the computed maximum acceleration at the crest is similar to that at the base, as previously indicated by the seismoscope records. The figure also shows the zones where the combination of initial static stresses and induced dynamic stresses would lead to the development of high pore-water pressures and large strains. In fact, the analysis showed that within the shaded area of the embankment the residual pore-water pressures would be equal to the overburden pressure and strains

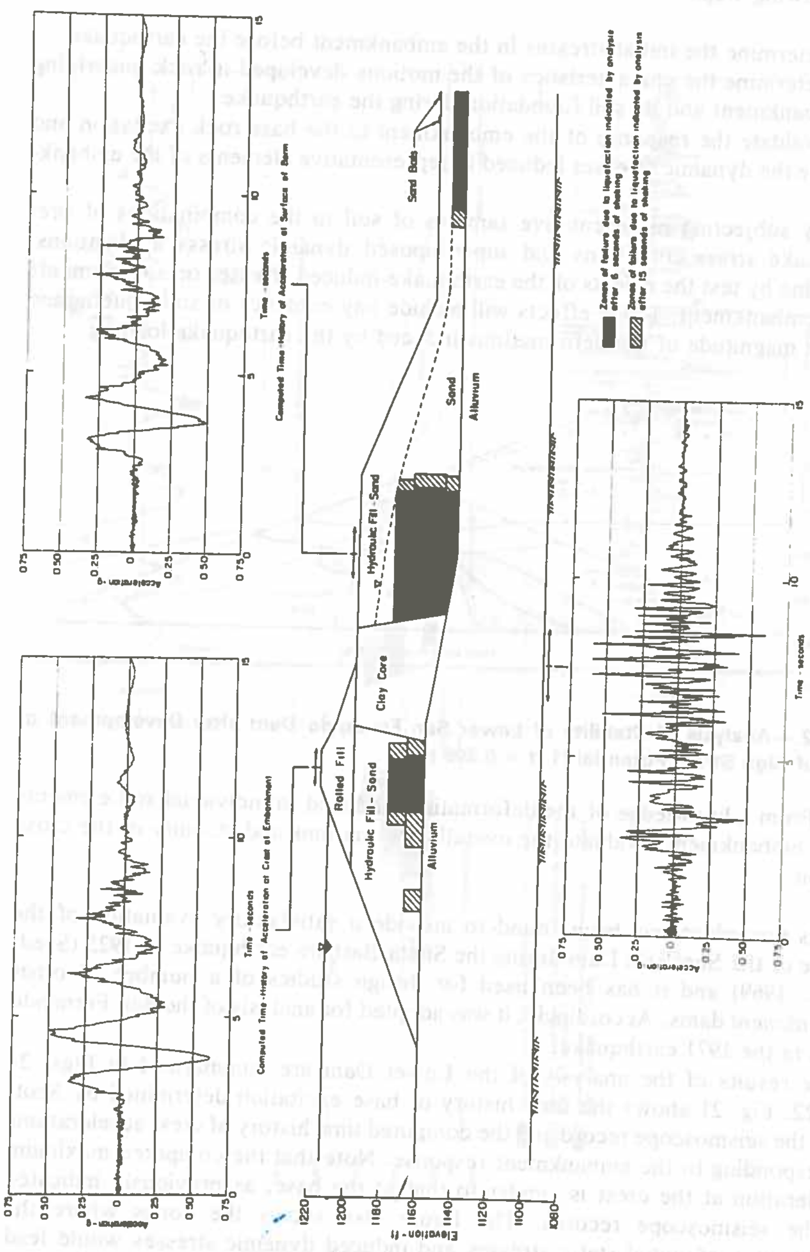


FIG. 23.—Analysis of Response of Upper Dam during San Fernando Earthquake to Base Motions Determined from Modified Pacoima Record (1 ft = 0.305 m)

ranging from 5%-30% would tend to occur. The zone in which shear strain would be particularly high (exceeding 15%) is shown in Fig. 22. If it is considered that the soil in this zone would provide no effective resistance to slide movements in the embankment (as a result of liquefaction and the development of high pore-water pressures during the earthquake), a stability analysis can be made to assess the stability of the slope against static failure, as shown in Fig. 22. Using average consolidated-undrained strength parameters because of the sudden change in stress distribution, the computed factor of safety along the critical sliding surface shown in Fig. 22 is about 1.06; using conservative values of strength parameters, the computed factor of safety is about 0.8.

These results would indicate that on completion of the ground motions or towards the end of the earthquake shaking, the upstream slope would be in a condition of incipient failure and might be expected to fail completely under the static weight of the embankment alone without the effect of inertia forces induced by the earthquake motions. Clearly, the same failure could occur in the later stages of the earthquake shaking once the zone of liquefaction and strength loss had developed. The extensive lateral movements of the slide mass were no doubt due in large part to a failure of this type, probably accompanied by a further loss in strength of the soil once the peak strength had been exceeded and some pore pressure increase in the outer shell due to the earthquake shaking.

Dynamic Analysis of Stability of Upper Dam during San Fernando Earthquake

Following exactly the same procedure as that described previously, a dynamic analysis was also made of the stability of the Upper San Fernando Dam (Seed, et al., 1973). However, because there was no record of rock motions in the vicinity of the dam available in this case, a modified form of the Pacoima record with a peak acceleration of 0.6g as shown in Fig. 23 was considered to be an appropriate representation of the probable base excitation.

The results of the analysis are summarized in Fig. 23 which shows the computed accelerations at the crest and downstream berm of the embankment and the zone in which liquefaction and strains exceeding 5% would be computed to occur. Following the same procedure as before and considering the zone where computed shear strains exceed 15% to make no contribution to the overall embankment stability, the factor of safety against a downstream slide was computed to be about 1.75. Thus, despite the extensive zone of liquefaction or high pore pressure developed by the earthquake shaking, the embankment would easily be able to withstand the small inertia forces developed later in the earthquake, together with the static stresses, without developing a residual downstream instability condition.

Analysis of the shear strains induced by the earthquake in the Upper Dam indicated an average shear strain potential of about 12%-16%, indicating a relative horizontal downstream movement of the crest and berm of about 4.5 ft-6 ft (1.4 m-1.8 m). This is in excellent accord with the observed downstream movement of the crest of about 5 ft (1.5 m). Clearly, this high degree of agreement is extremely fortuitous but it does indicate the potential of the analysis procedure for evaluating the stability and deformation of dams of this type.

Conclusions

The events associated with the performance of the Upper and Lower San

Fernando Dams during the earthquake of February 9, 1971 indicate that a major catastrophe was narrowly missed. Had any one of a number of possible conditions been slightly less favorable, such as the duration of shaking or the water level in the reservoir, the Lower Dam could have failed resulting in a sudden release of 10,000 acre-ft (12,300,000 m^3) of water over a heavily populated urban residential area. The immediate recognition by responsible authorities that the margin of safety was unacceptably close led to the investigation described in the preceding pages.

The main conclusions resulting from the study with regard to the location of the slide surface and the mechanics of sliding are summarized briefly.

Field Conditions and Mechanism of Failure

1. Construction of the Lower and Upper San Fernando Dams was initiated in the years 1912 and 1921, respectively, by hydraulic fill methods directly on the natural alluvial soils, with additional zones of compacted fill being added later.

2. The soil conditions at both dams were similar. The foundation alluvium consisted of a heterogeneous mixture of fairly well-graded clayey and sandy gravel with an average relative density of about 65%–70%. The hydraulic fill forming the shells of the dam consisted of stratified layers of coarse to fine sand and clay, the degree of stratification tending to decrease from the outside toward a central clay core. The general characteristics of these soils are shown in Table 1.

3. The earthquake of February 9, 1971 produced about 12 sec of strong shaking at the damsites with a peak acceleration of about 0.5g in the rock underlying the embankments.

4. As a result of this seismic shaking, the crest of the Upper Dam moved about 5 ft–6 ft (1.5 m–1.8 m) downstream and settled about 3 ft (0.9 m) while the upstream part of the embankment of the Lower Dam, including the upper 30 ft (9.2 m) of the crest, moved 70 ft (21 m) or more into the reservoir.

5. The slide movements in the upstream slope of the Lower Dam occurred as a result of an increase in pore-water pressure in the embankment soils resulting from the ground shaking and the resulting loss of strength and liquefaction of the hydraulic fill near the base of the embankment. Evidence supporting this conclusion includes: (a) Observed increases in pore-water pressure in the soils comprising the downstream part of the embankment; (b) the large horizontal displacements [about 75 ft (23 m)] of the main parts of the slide mass; (c) the spreading of embankment soil about 250 ft (76 m) beyond the toe of the embankment; (d) the formation of sand boils in the slide debris; (e) the formation of cracks filled with liquefied sand in the slide mass; (f) the complete distortion and intermixing of sand and clay layers in the slide zone; (g) the mechanism of failure, as evidenced by the location of slide debris, which was similar to that of other slides resulting from soil liquefaction; (h) the positions of hydraulic sand fill in the slide debris in some areas, which could only have resulted from the soil being in a state of liquefaction or complete strength loss during the slide movements; (i) reconstruction of the mechanics of sliding, which indicated an extensive zone of liquefaction near the base of the embankment; and (j) cyclic load tests on undisturbed samples consolidated to stress conditions

equivalent to those existing in the dam prior to the earthquake and subjected to cyclic stresses similar to those caused by the earthquake, which consistently produced failure by liquefaction where the pore-water pressure reached values equal to the confining pressure and large deformations developed thereafter.

6. A review of all available evidence, including a reconstruction of the embankment section and the results of dynamic analyses of embankment stability, strongly suggests the following mechanism of failure of the upstream shell of the Lower San Fernando Dam: After about 12 sec of strong shaking very high pore-water pressures had developed in an extensive zone of hydraulic fill near the base of the embankment and upstream of the clay core so that much of this soil was in a liquefied condition. At this stage, the shear resistance of the soil in the upstream shell could not withstand the dead-load stresses caused by the weight of the embankment and slide movements developed. The slide mass moved outwards on the liquefied soil, breaking into blocks as the movement developed, and removing support from the clay core, which was then extruded into the remaining part of the shell material by the pressure of the overlying portions of the embankment.

7. The horizontal movement of the Upper San Fernando Dam also resulted from increases in pore-water pressure in the embankment soils leading to some loss of strength and complete liquefaction in some zones of the embankment; however, because a significant body of the sand in the upstream and downstream shells retained considerably strength, complete failure could not occur and the movements were limited in extent.

8. There was no evidence of slide movements in the foundation alluvium for either dam; samples taken from the Lower Dam showed that the position of the original ground surface was unchanged by the slide movements immediately above, and careful examination of samples of alluvium at both damsites showed no evidence of disturbance. These observations are consistent with the laboratory and field determinations that the relative density of the alluvium at both sites was significantly higher than that of the hydraulic fill, and that the strength of the alluvium under cyclic loading conditions was significantly higher than that of the hydraulic fill under the same confining pressure conditions.

9. Note that three other hydraulic fill dams (Fairmont, Lower Franklin, and Silver Lake) were subjected to base rock motions with maximum accelerations of the order of 0.2g during the earthquake without any detrimental effects, indicating that hydraulic fill dams are not inherently unstable structures but only become so when they are subjected to shaking of sufficient intensity and duration to initiate large increases in pore-water pressure and accompanying loss of strength in the soil. Furthermore, another hydraulic fill dam (Dry Canyon), with no water in the reservoir, was subjected to shaking of comparable intensity to that at the Upper and Lower San Fernando damsites, without any detrimental effects, indicating that it is the presence of water pressure in the soils that leads to instability.

Pseudostatic Analyses

1. Pseudostatic analyses of the seismic stability of the embankment of the Lower Dam made several years before the earthquake using conservative strength values and a seismic coefficient of 0.15, together with a partial drawdown,

showed a factor of safety of about 1.05 for the upstream slope and led to the conclusion that the embankment would not fail due to anticipated earthquake ground motions. Using average strength values for the embankment soil and discounting any effects of drawdown, the computed factor of safety for the same seismic coefficient would have been about 1.5. However, the upstream slope failed completely during the San Fernando earthquake.

2. Pseudostatic analyses of the seismic stability of the embankment of the Upper Dam using average strength values for the embankment soil and a seismic coefficient of 0.15 show a computed factor of safety of about 2.0-2.5 depending on the method of data analysis. However, the embankment moved 5 ft-6 ft (1.5 m-1.8 m) downstream and apparently approached a condition of failure during the earthquake.

3. While pseudostatic analyses using a seismic coefficient of 0.15 were inadequate for predicting the stability of the embankments during the San Fernando earthquake, they did serve to indicate the locations of the most critical sliding surfaces in the embankment sections.

4. To predict the conditions of instability of the San Fernando dams during the earthquake of February 9, 1971, using the pseudostatic method of analysis, it would have been necessary to use a seismic coefficient of the order of 0.2-0.3 for the Lower Dam and about 0.5 for the Upper Dam.

5. The pseudostatic method of analysis with seismic coefficients of the order of 0.1-0.15 does not appear to provide an adequate basis for evaluating the seismic stability of dams that may fail as a result of increased pore-water pressures and severe loss in the embankment or foundation soils for the intensity of ground shaking that developed in the San Fernando earthquake.

Dynamic Response Analyses.—Dynamic analyses of the response of the San Fernando dams appear to provide a satisfactory basis for assessing the stability and deformations of the embankments during the earthquake. This type of analysis indicates the development of a zone of liquefaction along the base of the upstream shell of the Lower Dam, which would be sufficiently extensive near the end of the earthquake shaking to lead to a condition of instability (factor of safety <1). The same analysis procedure applied to the Upper Dam indicates that complete instability would not develop (factor of safety $\simeq 1.75$) but the development of large shear strains would lead to substantial deformations of the embankment section. The computed movements of the crest and berm of the embankment [4.5 ft-6 ft (1.4 m-1.8 m)] are in reasonable agreement with the observed downstream displacement of 5 ft (1.5 m) at the crest of the dam.

The slides in the San Fernando dams in the 1971 earthquake and the results of the investigations of them have had a profound effect on engineering evaluations of the seismic stability of similar dams in California. As a result of these events, the Division of Safety of Dams of the State Department of Water Resources has required owners of all hydraulic fill dams (29 in number) to reevaluate the safety of those dams using dynamic analysis procedures. In addition (Cortright, 1973), the Division

> is also looking at about 70 other dams, other than hydraulic fills, constructed before 1935 . . . which may contain loose susceptible soils in the embankment or foundation. Detailed engineering investigations for resistance to earthquakes are being required on all identified dams. The Division is planning also to look ultimately at all other existing dams using the currently available dynamic techniques wherever they are sufficiently important to warrant such attention.

As a result of these engineering studies, remedial measures are being required on a number of dams.

Similar studies are also being initiated on potentially vulnerable dams in other parts of the United States.

Because of their significance in the seismic design of earth dams, the slides in the San Fernando dams and the conditions producing them have been described in some detail in this paper. Further details are contained in the report on the slides (Seed, et al., 1973). In addition, companion papers will present information on the exploration of the characteristics of the soils comprising the dams and details of the dynamic analyses of the stability of the embankments in the hope that engineers concerned with seismic design problems will be able to draw maximum benefits from the experience provided by the San Fernando dam slides.

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DISCUSSION

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