Design and Performance of Arena Dam

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SYNOPSIS: Arena Dam is located in north-central Trinidad, West Indies. The dam forms a 35,000-acre-foot reservoir, which serves as the main raw water storage facility for Trinidad. The 1.6-million-cubic-yard earthfill embankment has a crest elevation 80 feet above the original streambed. The upstream-sloping core is composed of dispersive clay. The shells are composed of compacted fine sand and silty fine sand. The dam is founded on deep, stiff, fissured clay deposits interbedded with sand. The project is located approximately 12 miles from the El Pilar Fault, a major Caribbean fault with seismic activity comparable to that of the San Andreas Fault in the United States. Important design concerns included the dispersive clay core, residual strength properties of the foundation, embankment and control structure settlement, and the seismic environment. This paper discusses the design criteria and approach, and field performance data from foundation and embankment piezometers and survey monuments in the outlet conduit.

INTRODUCTION

Arena Dam is located on the Arena River in north-central Trinidad. Figures 1 through 4 show the location, plan view, and sections of the dam. The dam forms Arena Reservoir, which provides 35,000 acre-feet of storage to augment the dry season flow of the Caroni River. Water from the Arena River is impounded during the wet season (January through June) for release during the dry season (July through December). The Arena River provides about 40 percent of the water needed to fill the reservoir; the remainder is obtained from the adjacent Tumpuna River. A weir located downstream of the confluence of the Arena and Tumpuna Rivers backs water up the Arena River channel to the dam toe. During the wet season, a pump station at the base of the dam pumps water through the outlet works and into the reservoir. During the dry season, water is released through the outlet works and flows down the Arena and Tumpuna Rivers to the Caroni River, from which it is withdrawn for treatment. This system allows year-round production of 75 million U.S. gallons per day of treated drinking water, about half of the island's supply. The dam was designed from 1973 to 1975 and constructed from 1977 to 1983 for the Water and Sewerage Authority of the Government of Trinidad and Tobago.

PHYSIOGRAPHY AND GEOLOGY

The main physiographic features of Trinidad are three mountain ranges and two intervening basins. The Northern Range mountains are the largest mountains on the island. They are oriented east-west, and form the northernmost portion of the land mass. The Southern Range lies along the south coast of the island, and is also oriented east-west. The Central Range forms much of the central portion of the land mass, and is oriented generally west-southwest to east-northeast. The Caroni River Basin lies between the Northern and Central Ranges, and drains to the west. The Arena River rises in

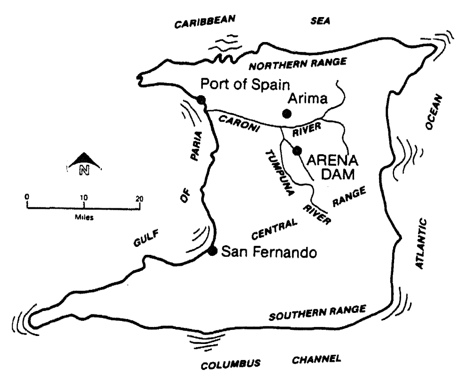

Fig. 1
VICINITY MAP

the Central Range near the center of the island, and flows northerly to the Caroni River.

At the southern toe of the Northern Range, and 12 miles north of the dam, lies the El Pilar Fault Zone. This major fault is the contact between the South American tectonic plate from the Caribbean subplate. The fault turns northward to the east of Trinidad. To the west, it extends into northern Venezuela. From a seismicity standpoint, the El Pilar fault system is a major fault comparable to California's San Andreas Fault. Other regional faults are present, but none are as significant as the El Pilar Fault due to both its energy release potential and its proximity to the dam. A number of small faults in the vicinity of the dam are believed to have resulted from the same forces that created the major fault systems. Few earthquakes have occurred in Trinidad, based on over 300 years of historical data; seismic activity is centered in northern Venezuela, 55 to 95 miles west of the dam.

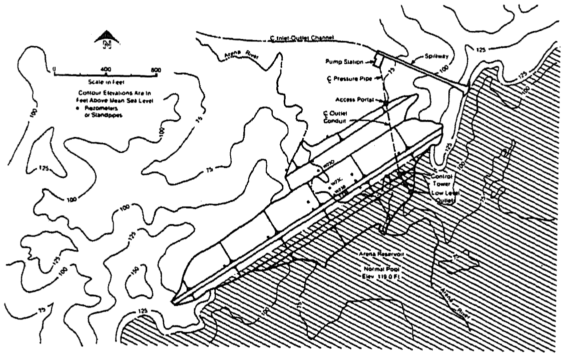

Fig. 2
PLAN OF DAM

At the damsite, a flood plain approximately 800 feet wide lies at an elevation of approximately 73 feet above mean sea level. This flood plain is underlain by up to 50 feet of alluvium. Prior to dam construction, the Arena River flowed in a channel approximately 20 feet deep incised in the flood plain. The reservoir area consists of rolling hills with relief generally ranging from 150 to 200 feet.

The reservoir site is underlain by a thick sequence of poorly indurated sandstones and claystones, and by alluvium. The claystones, known geologically as the Upper Caparo Clay, consist of highly consolidated, stiff, fissured, fat clays. Based on the regional geology and data from nearby oil wells, this clay is at least 800 feet thick at the dam. Approximately 2,400 feet of overlying material have been removed from the site by geologic forces.

SITE CONDITIONS

The Upper Caparo Clay contains frequent very thin sand partings; occasional thin layers of sand (up to 2 feet thick); interlayered sand, silt, and clay beds varying from a few inches to a few feet in thickness; and some varved sand/ clay layers. Typical upper Caparo clay properties are given in Table 1. The recompression index was determined by conducting conventional consolidation tests with repeated loading and unloading at the stress levels expected to occur after dam construction. The average slope of the unloading and reloading curves was used as the recompression index. The plasticity characteristics of the clay are shown in Figure 5. The grain size distribution is shown in Figure 6.

The Upper Caparo Clay is typically highly plastic. Kaolinite predominates, followed by illite and montmorillonite. Measured clay mineral percentages (x-ray diffraction method) ranged from 40 to 65 percent for kaolinite, 19 to 34 percent for illite, and 16 to 28 percent for montmorillonite.

The alluvium consists of interbedded soft clays and loose sands up to 50 feet thick. Standard

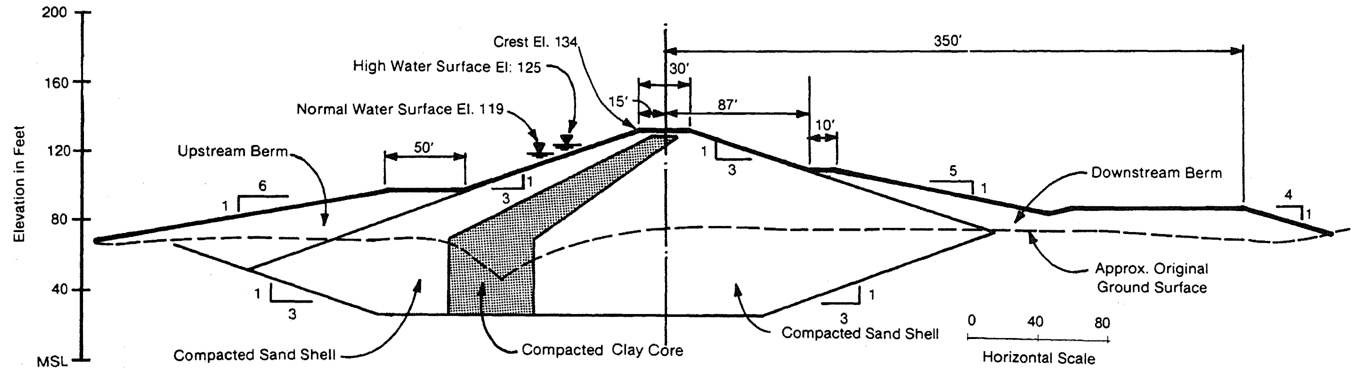

Fig. 3
MAXIMUM DAM SECTION

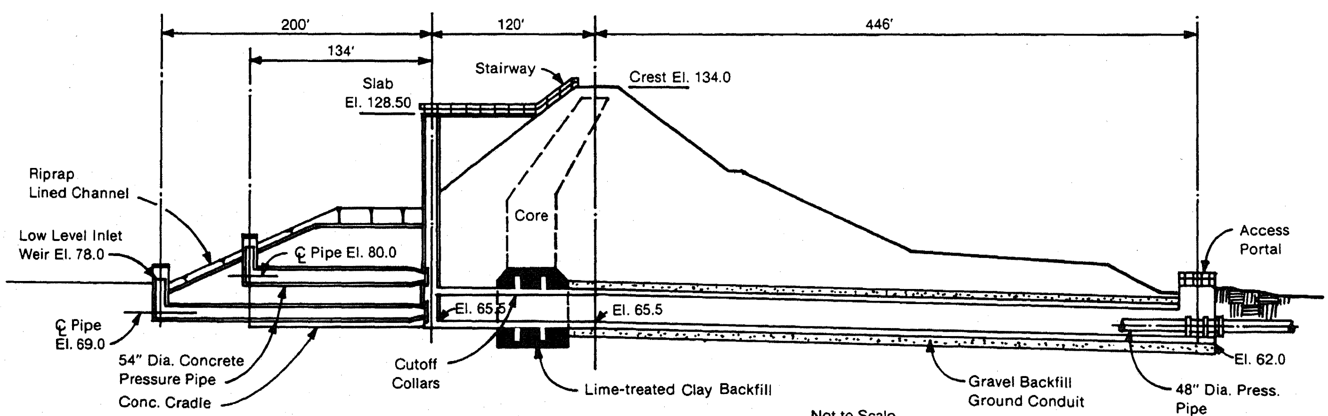

Fig. 4
OUTLET CONDUIT PROFILE

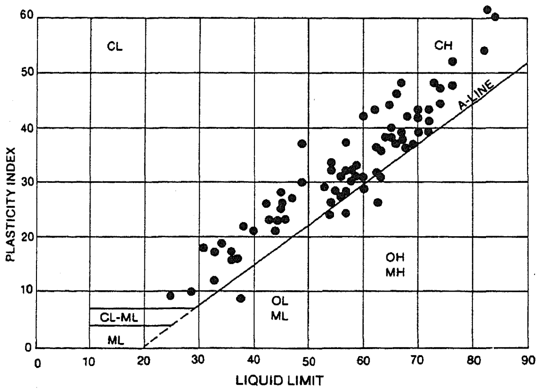

Fig. 5
UPPER CAPARO CLAY PLASTICITY

TABLE 1. Typical Engineering Properties of the Upper Caparo Clay

Parameter	Typical Values
Dry Unit Weight, pcf	85-110
Moisture Content, percent	20-35
Percent Passing #200 Sieve	75-100
In Situ Initial Void Ratio	0.71-0.93
Recompression Index	0.12

Penetration Test results in the alluvium ranged from 4 to 41. Most values were 20 or less; the higher values were generally associated with clayey sand lenses in the alluvium.

To the north of the damsite, the Mahaica or Arena Sand is exposed. This is a fine sand to fine silty sand. The grain size range is shown in Figure 6.

DESIGN PROPERTIES

Design properties were selected based on laboratory testing. The material properties chosen for design are presented in Table 2.

TABLE 2. Adopted Engineering Design Properties

Material Type	Total Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (degrees)
Remolded Borrow	120	200	19
Upper Caparo Clay:			
Peak strength	120	850	13
Residual strength	120	250	7.5
In-situ Alluvium	120	0	23
Arena Sand	125	0	28

MAJOR DESIGN CONCERNS

Seismicity

Because of the project's importance, its location upstream of major population centers, and its proximity to the El Pilar Fault Zone, a detailed study of the site's seismicity was undertaken by seismic consultants from the

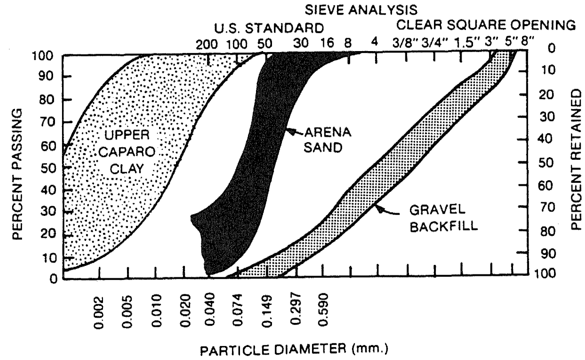

Fig. 6
GRAIN SIZE DISTRIBUTIONS

University of the West Indies (Tomblin and Aspinall, 1973). The results of the seismicity study were used to develop three design earthquakes (Table 3). These three design earthquake levels were developed in the early 1970's, and differ from current practice.

TABLE 3. Design Earthquake Accelerations

Acceleration Event	Hypocentral Distance (mi)	Maximum Horizontal (% of g)
MPE	94	18
MRE	56	60
MCE	12	160

The Maximum Probable Earthquake (MPE) was the greatest earthquake that was reasonably expected to occur during the project life. During the MPE, only minor damage would be tolerated, and all structures were designed to remain functional. Damages such as small cracks in concrete or minor surface slumps were considered to be acceptable.

The Maximum Credible Earthquake (MCE) was the greatest earthquake that could possibly be expected to occur if the greatest conceivable rupture were to occur at the closest possible location and at the shallowest possible depth, releasing the maximum conceivable amount of energy. This resulted in theoretical ground accelerations at the damsite that were so large that it was judged impractical to attempt to design a dam that could survive such an event.

Historic earthquakes had occurred 55 or more miles west of the damsite. Consequently, a third event, the Maximum Reasonable Earthquake (MRE) was defined as the earthquake that could be expected to occur if the greatest conceivable rupture were to occur 55 miles west of the site. The design goal for the MRE was to retain the reservoir, with the understanding that damage could occur at ancillary facilities such as the pump station or spillway.

Settlement

Foundation settlement calculations were made based on Terzaghi theory, except that the recompression index was used in place of the compression index. Stresses were assumed to be distributed elastically in the foundation in accordance with the Boussinesq equations. The predicted total settlement was 6 to 8 feet at the centerline of the dam at its maximum section. This estimated settlement was reduced by a factor of two based on the observation of Terzaghi and Peck (1967) that Terzaghi consolidation theory overestimates settlement of highly preconsolidated soils by a factor of 2 to 5 or more. The resulting predicted consolidation settlement was 3 to 4 feet. Drainage conditions were complex beneath the dam, ranging from thin laminations of sand to widely spaced sand beds. It was estimated that approximately one-half of the consolidation settlement would occur during the dam construction period. Postconstruction foundation consolidation was thus estimated to be 1.5 to 2 feet. The embankment itself was expected to undergo 0.3 to 0.4 foot of postconstruction settlement based on the data collected by the U.S. Bureau of Reclamation and reported in Sherard et al. (1963). Thus, crest settlement of approximately 2 to 2.5 feet was anticipated after construction.

The same method was used at the outlet conduit. Settlement during embankment construction was included since the outlet conduit was constructed prior to the embankment. The maximum estimated settlement of the outlet conduit was approximately 3.5 feet at the dam centerline.

Base Spreading

Dam settlement was expected to cause spreading of the dam base, as described by Rutledge and Gould (1973). The principal concern was the outlet conduit. The maximum predicted extension was approximately 1/2 inch per 20 feet at the dam centerline. Predicted extensions decreased toward the dam toes. The outlet conduit had to be designed to tolerate this extension as well as the predicted settlement.

Dispersive Clay

During the dam design, dispersive clays were becoming widely recognized as a potential problem in dam design and construction (Sherard et al., 1972). Several of the preliminary identification tests (crumb test, SCS laboratory dispersion test, cation exchange capacity) were performed on samples of soil from the

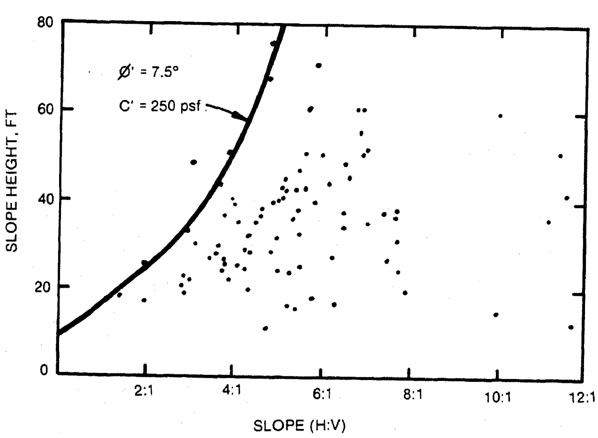

Fig. 7
NATURAL SLOPES NEAR DAM

dam area. When these tests suggested that dispersive clay could be present, a more definitive test (the pinhole test, as later described by Sherard et al., 1976) was performed on a variety of samples. The pinhole test results showed that most of the soils in the dam and reservoir area were dispersive. Thus, the design was required to provide defense against piping erosion of the dam and foundation materials.

Stiff, Fissured Clay Foundation

Laboratory testing of undisturbed and remolded specimens of the Upper Caparo Clay exhibited high peak strength at low strain followed by a rapid strength reduction until a constant value was reached. This suggested that residual strength analysis similar to that used in considering progressive failure (Bjerrum, 1967) should be used. Residual strengths were determined by slow direct shear tests and confirmed by a review of natural slopes.

Laboratory residual strength values were determined by conducting direct shear tests on 2.5-inch-diameter specimens with precut shear surfaces at approximately 0.0001 in/min. The results, shown in Table 2, were used for the stability analysis.

For the natural slope review, 66 slopes near the damsite were measured with tape and hand level. An additional 19 slopes were measured on topographic maps. The data were plotted as shown in Figure 7. A bounding curve was then drawn, representing the maximum observed slope for any given height. By assuming that the safety factor of these slopes was approximately one during saturated conditions, effective cohesion and friction angle values could be calculated using stability charts (Taylor, 1948). An effective cohesion of 200 pounds per square foot and an effective friction angle of 7.5 degrees produced a bounding curve that enveloped all but one of the observed points with a smooth curve, as shown.

DESIGN APPROACH

The above concerns required that special details be incorporated into the design of the dam and outlet conduit.

Stability Analysis

Because of the potential for progressive failure of the foundation, slope stability analysis for long-term conditions considered both peak and residual foundation strengths. The stability design criteria involved a long-term safety factor of not less than 1.5 using peak strengths, and not less than 1.1 using residual strengths. The calculated safety factor for the residual foundation strength case was 1.1, which controlled the design. The calculated safety factor was 2.0 using peak foundation strenths parameters.

Dam Section

The dam section is shown in Figure 3. An upstream-sloping core was chosen to provide a large drained downstream shell for improved stability under earthquake conditions. The core was constructed of dispersive clay obtained from the reservoir area. The clay was compacted to not less than 95 percent of Standard Proctor density at a moisture content between 1 percentage point dry and 5 percentage points wet of optimum. This was provided a flexible, low permeability core. With the large settlements expected and the possibility of distortion due to earthquakes, the core was made more flexible and the dam crest was made wider and higher than it would have otherwise been.

The dam shells were constructed of fine Arena Sand. The specifications required a minimum of six complete coverages of each 6-inch-thick sand lift with a vibratory sheepsfoot roller having a dynamic energy of at least 25,000 pounds. The sand moisture content was not permitted to exceed 3 percentage points wet of Standard Proctor optimum. If, after six passes of the vibratory roller, a density less than 98 percent of Standard Proctor Density was obtained in any area, additional passes of the specified roller were paid for on an hourly basis. Densities generally exceeded 98 percent of Standard Proctor density with six passes.

A 20-foot-wide zone immediately downstream of the core was constructed of sand selected to provide sand with not more than 8 percent, by weight, passing the No. 200 sieve, washed basis. This zone serves as a downstream filter for the dispersive clay core and as a drain. Laboratory filter tests in both the pinhole apparatus and in 4-inch-diameter molds were used to confirm that the sand would filter the clay, even when preformed holes were made in the clay.

The upstream shell sand provides a supply of cohesionless material to flow into cracks that may develop in the core due to settlement, earthquake, or other causes.

Outlet Conduit

The outlet conduit (Figures 4 and 8) was designed as a reinforced concrete cut-and-cover dry tunnel with a 48-inch-diameter steel pressure pipe. This design was chosen to accommodate movement of the pressure pipe and to permit access for inspection and maintenance of the conduit.

The outlet conduit was constructed in 20-foot-long segments. Construction joints were provided on 20-foot centers so that the conduit would elongate in a controlled manner. Because of the area's seismicity and the expected dam base spreading, the longitudinal reinforcement was extended through the construction joints to provide continuous conduit reinforcement. A 3-foot-long plastic pipe sleeve was provided for each bar crossing each construction joint so that extension of the conduit would result in opening of the joints rather than cracking of the conduit segments. To provide for this extension, high-ductility steel was used for longitudinal reinforcing.

Each conduit construction joint was provided with a center-bulb waterstop to allow extension without leakage. The conduit was surrounded with a 2-foot-thick zone of 2-inch-minus, well-graded, sandy gravel (Figure 6). This gravel acts as a filter for the sand, and is

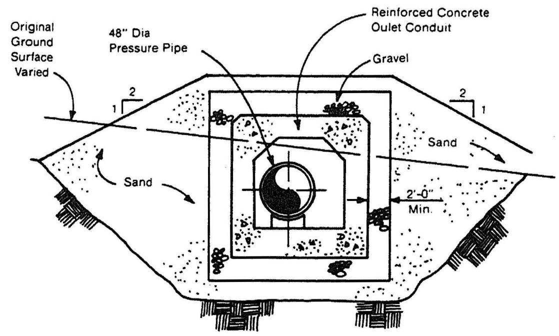

Fig. 8
OUTLET CONDUIT SECTION

coarse enough to prevent piping into the predicted joint openings and other cracks that may develop. The gravel also provides drainage for the downstream shell near the conduit.

The 48-inch-diameter steel pressure pipeline was constructed in 40-foot lengths with thrust-restrained flexible couplings. The pipe joints were located at the conduit joints so the conduit and the pipe could move together.

At the core penetration, the clay core material within 5 feet of the outlet conduit in all directions was treated with hydrated lime [Ca(OH)2]. Pinhole tests conducted on soil samples treated with lime showed that an application rate of 3 percent by dry weight of soil rendered the soil nondispersive. This reduced the potential for piping of the clay core into cracks or joints in the conduit.

Control Tower

The control tower was located in the upstream shell of the embankment (Figure 4) to improve the tower's seismic performance by reducing the height of the free-standing portion of the tower.

Because of the tower's location within the embankment, considerable settlement was anticipated. The estimated total settlement at the tower location was 2.5 to 3 feet. For this reason, the uppermost lift of the tower concrete was not placed until the dam was topped out in the area of the tower. This reduced the estimated postconstruction settlement of the tower top by eliminating the effects of elastic settlement and a portion of the consolidation settlement.

A small amount of tower tilting was also expected during and after construction because of the settlement and spreading of the dam. This tilting, together with movement during earthquakes, was expected to cause several inches of relative motion between the tower and the dam crest. To maintain access to the tower, the bridge was connected to the tower with a structural hinge, and was supported on the dam with a special bridge bearing that allows the bridge to move up to 18 inches toward or away from the dam crest.

EMBANKMENT PERFORMANCE

The embankment performance was satisfactory and similar to the design assumptions. Figure 9

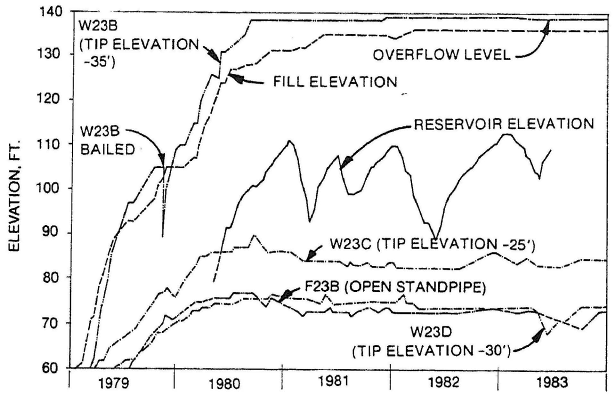

Fig. 9
SELECTED EMBANKMENT PIEZOMETERS

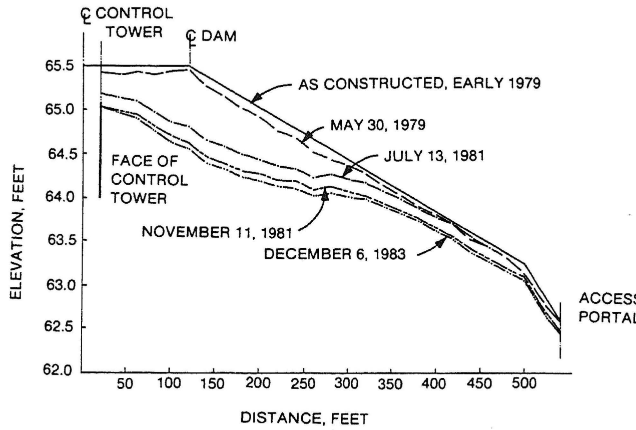

Fig. 10
OUTLET CONDUIT SETTLEMENT

shows the results of pore pressure monitoring at four instruments and the rate of embankment construction at the instruments. The instrument locations are shown in Figure 2.

The data shown are typical of 12 piezometers and 6 foundation settlement plates/standpipes. Only one piezometer showed unusual behavior: W23B. This piezometer was installed by drilling approximately 50 feet into the foundation from the bottom of the core trench prior to embankment construction. The riser was then protected and extended throughout embankment construction. At this piezometer, pore pressures rose as the dam was constructed. Eventually, the water level was above the dam crest. The riser pipe was extended above the crest, but overflow continued. Excess foundation pore pressure had been anticipated during design, but the dam stability in this area was reanalyzed during construction based on the results from piezometer W23B using a variety of conservative pore pressure assumptions. Safety factors remained acceptable, so construction was not slowed or stopped, and the design was not modified.

OUTLET CONDUIT PERFORMANCE

To monitor movement of the outlet conduit, monuments were installed inside the conduit and regularly surveyed. Figure 10 presents the settlement data for the outlet conduit. Despite the settlement, the outlet conduit continues to have positive drainage. The settlement at the dam centerline through December 1983 was approximately 1 foot, compared to the estimate of 3.5 feet.

CONCLUSIONS

Based on the performance of Arena Dam through 1983, the following conclusions are made:

1. Dispersive clay can be utilized in earth dams if proper precautions are taken.

2. The use of the recompression index in the terzaghi consolidation equations significantly overestimated settlement, even though predicted settlements had been halved to account for reported overestimates using the equations on preconsolidated clays.

3. Residual strength analysis resulted in a design that has performed successfully to date and allowed construction on a difficult site.

REFERENCES

Bjerrum, L. (1967), "Progressive Failure in Slopes of Overconsolidated Plastic Clay and Clay Shales," Jour. Soil Mech. and Foundations Div. ASCE, Vol. 93, No. SM5, September 1967, pp. 3-49. (Reprinted in Terzaghi Lectures, 1963-1972, ASCE, 1974.)

Rutledge, P. C., and J. P. Gould (1973), "Movements of Articulated Conduits Under Earth Dams on Compressible Foundations," in Embankment-Dam Engineering, Casagrande Volume, R. C. Hirschfeld and S. J. Poulos, eds., John Wiley and Sons, New York, pp. 209-237.

Sherard, J. L., R. S. Decker, and N. L. Ryker (1972), "Piping in Earth Dams of Dispersive Clay," Proc. Spec. Conf. on Performance of Earth and Earth-Supported Structures, ASCE, pp. 589-626.

Sherard, J. L., L. P. Dunnigan, R. S. Decker, and E. F. Steele (1976), "Pinhole Test for Identifying Dispersive Soils," Jour. Geot. Engrg. Div. ASCE, Vol. 102, No. GT1, January 1976, pp. 69-85.

Sherard, J. L., R. J. Woodward, S. F. Gizienski, and W. A. Clevenger (1963), Earth and Earth-Rock Dams, John Wiley and Sons, New York.

Taylor, D. W. (1948), Fundamentals of Soil Mechanics, John Wiley and Sons, New York.

Terzaghi, K., and R. B. Peck (1967), Soil Mechanics in Engineering Practice, Second Edition, John Wiley and Sons, New York.

Tomblin, J. F., and W. P. Aspinall (1973), "Seismic Risk at the Arena Dam Site, Trinidad" (unpublished).